Anderson Park Hotel

Northeast Corner of Redmond Way and 166th Avenue NE, Redmond, Washington

Preliminary Storm Drainage Report

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Prepared: July 2016
Revised: October 2016

TABLE OF CONTENTS

PROJECT DESCRIPTION

The site consists of a five separate parcels with a combined area of 1.22 acres. The site is currently developed, and existing buildings are present on four of the five parcels. The northwest parcel contains a fitness center, the southwest parcel contains a pizza restaurant, the southeast parcel contains a shops building with multiple uses, including an eatery and a nail salon, and the northeast parcel contains a single-family residence.

The project will involve demolition of existing buildings, pavement, and other site features, and construction of a multi-story hotel building with surface parking as well as a subterranean parking level, and associated driveways, stormwater management facilities, utilities, and landscaping.

The site is bordered to the west by 166th Avenue NE, to the south by Redmond Way, to the north by NE 79th Street, and to the east by existing developed properties.

This preliminary storm drainage report addresses the storm water management system for the entire site.

Site Location

A vicinity map showing the property location is provided as **Figure 1** in **Appendix A**.

Location: Northeast Corner of Redmond Way and 166th Avenue NE

Section: 12

Township: 25 North

Range: 5 East of W.M.

Parcel Numbers: 00580700000606

City, County, State: City of Redmond, King County, Washington State

Governing Agency: City of Redmond

Design Criteria: Washington State Department of Ecology Stormwater Management Manual for Western Washington, 2005, as modified by the City of Redmond Clearing, Grading, and Stormwater Management Technical Notebook, 2012.

DESIGN CRITERIA

The storm water management system has been designed in accordance with the Washington State Department of Ecology Stormwater Management Manual for Western Washington, 2005, as modified by the City of Redmond Clearing, Grading, and Stormwater Management Technical Notebook, 2012.

Table 1 summarizes the City of Redmond stormwater requirements.

Table 1 - City of Redmond Requirements	
Peak Run-off Control:	Match the pre-developed discharge rates from 50% of the 2-year peak flow up through the full 50-year peak flow.
Water Quality:	91^{st} percentile, 24-hour runoff rate
Detention:	As needed to comply with Peak Run-off Control Requirements.
Conveyance Design:	10-year storm event. 50-year storm event if upstream of detention.
Hydrological Design Method:	Western Washington Hydrologic Model for water quality and water quantity analysis.

The minimum requirements for stormwater management as outlined in the Department of Ecology's Stormwater Management Manual have all been addressed as follows:

Minimum requirement #1: Preparation of Stormwater Site Plans

This project proposes to replace greater than 2,000 square feet of impervious surface; thus, Minimum requirement #1 applies and a Stormwater Site Plan must be prepared for review by the local jurisdiction.

This Storm Drainage Report has been prepared to address this requirement.

Minimum requirement #2: Construction Stormwater Pollution Prevention

All erosion and sediment control measures shall be governed by the requirements of Department of Ecology's 2005 Stormwater Management Manual for Western Washington as modified by the City of Redmond Clearing, Grading, and Stormwater Management Technical Notebook, 2012, and the General Permit for Construction Stormwater. The twelve elements as identified in the Manual and provided below will be incorporated into the TESC plans

Element 1: Mark Clearing Limits
Element 2: Establish Construction Entrance
Element 3: Control Flow Rates
Element 4: Install Sediment Controls
Element 5: Stabilize Soils
Element 6: Protect Slopes
Element 7: Protect Drain Inlets
Element 8: Stabilize Channels and Outlets
Element 9: Control Pollutants
Element 10: Control De-watering
Element 11: Maintain BMPs
Element 12: Manage the Project

In addition, a National Pollutant Discharge Elimination System (NPDES) Permit will be obtained prior to construction and a Stormwater Pollution Prevention Plan (SWPPP) will be prepared for this project using the Department of Ecology template and Client standards.

Minimum requirement #3: Source Control of Pollution
The SWPPP prepared for this project will provide details and general guidance to utilize the Best Management Practices (BMP's) for source control of pollution. Refer to the Erosion Control section below for more detailed information regarding these BMP's.

Minimum requirement #4: Preservation of Natural Drainage Systems and Outfalls
Presently, on-site stormwater is collected in catch basins and piped to the public system, with connections in Redmond Way, 166th Ave. NE, and NE 79th St. Ultimately all the stormwater drains to an existing regional detention/treatment facility. The proposed storm drainage system will connect to the same existing system and eventually the same regional facility, thus preserving the natural drainage system.

Minimum requirement #5: On-site Stormwater Management
Stormwater management BMPs will be installed as appropriate to manage on-site stormwater. After completion of construction, all new landscaped areas within the project site will have compost amended soil in place per City of Redmond requirements.

Minimum requirement #6: Runoff Treatment
Water quality for the project will be provided at a regional facility.

Minimum requirement #7: Flow Control
Flow control for non-roof areas of the project will be provided at a regional facility. Runoff from roof areas will be directed to an on-site infiltration facility, which will provide 100% infiltration.

Minimum requirement #8: Wetlands Protection
There are no wetlands within the development area of the site.

Minimum requirement #9: Operation and Maintenance
The Operations and Maintenance of the storm water management system is detailed in the Operations and Maintenance section of this report.

EXISTING CONDITIONS

The site is currently developed, and existing buildings are present on four of the five parcels. The northwest parcel contains a fitness center, the southwest parcel contains a pizza restaurant, the southeast parcel contains a shops building with multiple uses, including an eatery and a nail salon, and the northeast parcel contains a single-family residence. The site is generally flat, with slopes typically ranging from 1% to 5%, although there are isolated areas of greater slopes in driveways and landscaped areas. The site is generally sloped from the northeast to the southwest.

The site is bordered to the west by 166th Avenue NE, to the south by Redmond Way, to the north by NE 79th Street, and to the east by existing developed properties. Presently, on-site stormwater is collected in catch basins and piped to the public system, with connections in Redmond Way, 166th Ave. NE, and NE 79th St. Ultimately all the stormwater drains to an existing regional detention/treatment facility.

The project site is located in a critical area designated as Wellhead Protection Zone 1 by the City of Redmond. The development shall conform to City of Redmond Municipal Code 21.64.050, which identifies requirements for developments in this critical area to mitigate the potential for introduction of harmful materials into the groundwater.

Table 2 – Existing Conditions					
Area	**Roof (ac)**	**Parking (ac)**	**Sidewalk (ac)**	**Landscape (ac)**	**Total (ac)**
Site	0.27	0.66	0.03	0.26	1.22
Upstream Run-on	0.00	0.00	0.00	0.00	0.00
Total Area	**0.27**	**0.66**	**0.03**	**0.26**	**1.22**

Soil Conditions

A detailed geotechnical study of the site was performed by Golder Associates to determine the properties of the surface and subsurface soils. Results of the investigations are documented in the Geotechnical Engineering Report prepared by Golder Associates dated January 2016, provided (without Appendices) in **Appendix C**.

Infiltration Rates

Per the geotechnical study performed by Golder Associates, on-site soils exhibited high infiltration rates typical of the shallow alluvial soils. A long-term design infiltration rate of 7 inches/hour was recommended.

Water Table

Per the geotechnical study performed by Golder Associates, the seasonal groundwater elevation in the vicinity of the site ranges from a low of about 26 feet to a high of 32.5 feet. Groundwater was encountered in the site borings, and was typically found at depths of 16 feet below grade or deeper. A conservative groundwater elevation of 34 feet was recommended.

DEVELOPED CONDITIONS

The project will involve demolition of existing buildings foundations, pavement, and other site features, and construction of a multi-story hotel, and associated surface and subterranean parking, driveways, stormwater management facilities, utilities, and landscaping. A proposed drainage plan is included in **Appendix B**.

Per the City of Redmond Clearing, Grading, and Stormwater Management Technical Notebook (2012) section 2.5.5, infiltration is prohibited for non-single family residential projects located in Wellhead Protection Zone 1. The intent of this regulation is to prevent pollutant-laden runoff from infiltrating into the groundwater. Since the project roof areas are non-pollutant-generating, runoff from these areas will need to be infiltrated on-site.

An open-bottom infiltration vault will be installed along the northern edge of the property, and will receive runoff from the roof areas. The system is designed to infiltrate 100% of the roof runoff, although an overflow pipe connecting to the existing system will be constructed to provide an emergency release.

The drainage concept for the project site is a network of curbs, gutters, catch basins, and underground pipes that collect surface water runoff throughout the site. The runoff will then be conveyed to the public storm drainage system where it is directed to the regional stormwater management facility.

Table 3 – Developed Conditions

Area	Roof (ac)	Parking (ac)	Sidewalk (ac)	Landscape (ac)	Total (ac)
Site	0.64	0.34	0.19	0.05	1.22
Upstream Run-on	0.00	0.00	0.00	0.00	0.00
Total Area	**0.64**	**0.34**	**0.19**	**0.05**	**1.22**

OFFSITE ANALYSIS

Upstream Analysis

Runoff for offsite areas to the south, west, and north are collected in the roadways, and do not flow onto the site. The adjacent property to the east does not appear to flow onto the site, and stormwater runoff from the site is assumed to be managed independently. Therefore, no upstream issues are anticipated.

Downstream Analysis

The existing project site is currently developed and largely consisting of impervious areas. Although the amount of impervious area will likely increase, the amount will be relatively minor. The existing storm drain trunk line in Redmond Way was designed to accommodate flows to the regional detention/treatment facility, and likely has adequate capacity to accommodate the minor increase in runoff. Therefore, no downstream issues are anticipated.

CONVEYANCE

On-site storm water conveyance has been calculated through gravity flow analysis of the piping network. Based on a 50-year storm event, peak runoff was routed through the system and determined to be adequate. A Uniform Flow Analysis utilizing Manning's equation was employed with a Manning's "n" value of 0.012.

Manning's Equation: $$Q = \frac{1.49}{n} xAxR^{2/3} xS^{1/2}$$

With: Q = Flow (cfs)
n = Manning's Roughness Coefficient (0.012)
A = Flow Area (sf)
R = Hydraulic Radius = Area/Wetted Perimeter (lf)
S = Slope of the pipe (ft/ft)

Conveyance calculations will be provided in a future submittal.

WATER QUALITY

Water quality treatment for non-roof project site will be provided at a regional facility.

Water quality treatment for roof areas is not required, as those areas are not pollutant-generating and will fully infiltrate on site.

DETENTION

Flow control for runoff from non-roof project areas will be provided at a regional facility.

Flow control for runoff from roof areas will be provided via an open-bottom infiltration vault located along the northern edge of the property. The vault is designed to provide 100% infiltration, although an overflow pipe connecting to the existing storm drainage system to provide an emergency release.

LOW IMPACT DEVELOPMENT (LID)

The majority of the development will be utilized as building or parking (with a subsurface parking garage). Because of this, there is inadequate horizontal space to construct LID BMPs to manage on-site stormwater. However, infiltration will be utilized as much as feasible; and open-bottom vault located in the portion of the site between the parking garage and the public right-of-way. This facility will fully infiltrate runoff generated from the roof areas of the project.

EROSION CONTROL

All erosion and sediment control measures shall be governed by the requirements of Department of Ecology's 2005 Storm Water Management Manual for Western Washington as modified by the City of Redmond Clearing, Grading, and Stormwater Management Technical Notebook, 2012, and the General Permit for Construction Storm water. A National Pollutant Discharge Elimination System (NPDES) Permit will be obtained and a Stormwater Pollution Prevention Plan (SWPPP) will be prepared for this project.

The proposed development includes an erosion/sedimentation control plan designed to prevent sediment-laden run-off from leaving the site during construction. The erosion potential of the site is influenced by four major factors: soil characteristics, vegetative cover, topography, and climate. Erosion/sedimentation control is achieved by a combination of structural measures, cover measures, and construction practices that are tailored to fit the specific site.

Prior to the start of any grading activity on the site, all erosion control measures, including installation of a stabilized construction entrance, shall be installed in accordance with the construction documents.

Best construction practices will be employed to properly clear and grade the site and to schedule construction activities. The planned construction sequence for the construction of the site is as follows:

Construction Sequence and Procedure

The Contractor will be responsible for implementing the following erosion control and storm water management control measures. The Contractor may designate these tasks to certain subcontractors as they see fit, but the ultimate responsibility for implementing these controls and ensuring their proper functioning remains with the Contractor. The order of activities will be as follows.

Phase 1

1. Prior to beginning any work on the project site a pre-construction conference must be held, and shall be attended by the general contractor, the project engineer, representatives from the affected utilities and a representative from the City of Redmond.
2. Mark clearing limits.
3. Install inlet protection to all existing catch basins.
4. Install temporary stabilized construction entrance.

5. Install perimeter silt fences, interceptor swales, etc. Protect existing and proposed infiltration areas.
6. Demolish existing buildings.
7. Protect and stabilize slopes.
8. Begin clearing and grubbing operations. Clearing and grubbing shall be done only in areas where earthwork will be performed and only in areas where construction is planned to commence within 7 days after clearing and grubbing between May 1 and September 30 or 2 days between October 1 and April 30.
9. Commence site grading.

Phase 2

1. Disturbed areas of the site where Construction Activity has ceased for more than 7 days between May 1 and September 30 or 2 days between October 1 and April 30 shall be temporarily seeded and watered.
2. Install haul road.
3. Construct building pad and install concrete wash out area.
4. Construct permanent storm water facilities.
5. Install utilities, underdrains, storm sewers, curbs and cutters.
6. Install inlet/outlet protection at the locations of all grate inlets, curb inlets, and at the ends of all exposed storm sewer pipes.
7. Install rip rap around outlet structures.
8. Prepare site for paving. Finalize pavement subgrade preparation.
9. Remove inlet protection around inlets and manholes no more than 48 hours prior to placing stabilized base course.
10. Install base material as required for pavement. Pave site. Do not pave over catch basins.
11. Complete final grading in non-parking areas and install permanent seeding and planting.
12. Remove silt fencing only after all paving is complete and exposed surfaces are stabilized.
13. Remove temporary construction exits only prior to pavement construction in these areas (These areas are to be paved last).

Temporary Soil Stabilization

Temporary stabilization practices for this project include:

- Temporary seeding and planting of all unpaved areas using the hydro-mulching grass seeding technique.
- Mulching exposed areas.
- Installation of Rolled Erosion Control Products.

Structural practices for this project include the following. Refer to the Erosion Control plans for specific locations and details:

- Inlet protection using fiber fabric.
- Outlet protection (velocity dissipation) using rip rap.
- Perimeter protection using silt fences.
- Stabilized construction entrance/exit points and staging area.
- Temporary sediment basin.
- Rock check dam.
- Silt fence.
- Interceptor swales.
- Temporary storm drain riser.

Daily inspection of the erosion control measures will be required during construction. Any sediment buildup shall be removed and disposed offsite at an appropriate disposal facility.

Vehicle tracking of mud off-site shall be avoided. A gravel construction entrance/exit will be installed at a location to enter the site. The construction entrance/exit is a minimum requirement and may be supplemented if tracking of mud onto public streets becomes excessive. In the event that mud is tracked off site, it shall be swept and disposed of offsite on a daily basis. Depending on the amount of tracked mud, a vehicle road sweeper may be required.

Because vegetative cover is the most important form of erosion control, construction practices must adhere to stringent cover requirements. More specifically, the contractor will not be allowed to leave soils open for more than 7 days between May 1 and September 30 and 2 days between October 1 and April 30, and in some cases, immediate seeding or mulching will be required. Areas next to paved areas may be armored with crushed rock sub-base in place of other stabilizing measures.

Permanent Erosion Control and Site Restoration

Upon completion of the project, areas of the site that are not stabilized with paving, rooftops, or landscaping as shown on the site plans will be protected with either grass, ground cover/plantings or existing vegetation as shown on the Landscape Plans. In general, storm runoff from the site will be collected by catch basins connected to a storm water quality structure and then flows into detention system.

Inspection Sequence

Inspections are required at least every seven (7) days and within 24 hours following any rainfall and shall continue until the site complies with the Final Stabilization section of this document. The ESC Lead shall be the responsibility of the contractor.

Week # Date: *To be inspected daily	ESC Status: Ng = No good Ok = Okay Fx = Fixed						If Status is "No Good" describe failure	If failure was "Fixed", describe solution
	M	T	W	T	F	S		
*Silt Fence– Site Perimeter intact?								
*Silt Fence – Infiltration Pond intact?								
*Construction Entrance–Sediment in Street?								
Storm Drain Inlets – Sediment Buildup?								
Storm Drain Inlets – Protection Intact?								
Soil Excavation – Monitor for contaminants								
Soil Stockpiles – Stabilized?								
Soil Stockpiles – Visible Erosion/Rills?								
Swales/check dams – Swales Stabilized?								
Swales/check dams – Check Dams Intact?								
Sediment Pond – Sediment Buildup?								
Sediment Pond – Riser Intact?								

Control of Pollutants Other Than Sediments

Pollutants shall be controlled on the work site through the utilization of a centralized area for equipment, a concrete truck washout, and an area designated for temporary storage of debris and stockpiled materials.

OPERATIONS AND MAINTENANCE

The owner or operator of the project shall be responsible for maintaining the stormwater facilities in accordance with local requirements. Proper maintenance is important for adequate functioning of the stormwater facilities. The following maintenance program is recommended for this project:

Maintenance Checklist for Catch Basins and Inlets

Frequency	Drainage System Feature	√	Problem	Conditions To Check For	Conditions That Should Exist
M,S	General		Trash, debris, and sediment in or on basin	Trash or debris in front of the catch basin opening is blocking capacity by more than 10%.	No trash or debris located immediately in front of catch basin opening. Grate is kept clean and allows water to enter.
M				Sediment or debris (in the basin) that exceeds 1/3 the depth from the bottom of basin to invert of the lowest pipe into or out of the basin.	No sediment or debris in the catch basin. Catch basin is dug out and clean.
M,S				Trash or debris in any inlet or pipe blocking more than 1/3 of its height.	Inlet and Outlet pipes free of trash or debris.
M			Structural damage to frame and/or top slab	Corner of frame extends more than ¾ inch past curb face into the street (if applicable).	Frame is even with curb.
M				Top slab has holes larger than 2 square inches or cracks wider than ¼ inch (intent is to make sure all material is running into the basin).	Top slab is free of holes and cracks.
M				Frame not sitting flush on top slab, i.e., separation of more than ¾ inch of the frame from the top slab.	Frame is sitting flush on top slab.
A			Cracks in basin walls/bottom	Cracks wider than ½ inch and longer than 3 feet, any evidence of soil particles entering catch basin through cracks, or maintenance person judges that structure is unsound.	Basin replaced or repaired to design standards. Contact a professional engineer for evaluation.
A				Cracks wider than ½ inch and longer than 1 foot at the joint of any inlet/outlet pipe or any evidence of soil particles entering catch basin through cracks.	No cracks more than ¼ inch wide at the joint of inlet/outlet pipe. Contact a professional engineer for evaluation.
A			Settlement/ misalignment	Basin has settled more than 1 inch or has rotated more than 2 inches out of alignment.	Basin replaced or repaired to design standards. Contact a professional engineer for evaluation.
M,S			Fire hazard of other pollution	Presence of chemicals such as natural gas, oil, and gasoline. Obnoxious color, odor, or sludge noted.	No color, odor, or sludge. Basin is dug out and clean.
M,S			Outlet pipe is clogged with vegetation	Vegetation or roots growing in inlet/outlet pipe joints that is more than 6 inches tall and less than 6 inches apart.	No vegetation or root growth present.

Maintenance Checklist for Conveyance Systems (Pipes, Ditches and Swales)

Frequency	Drainage System Feature	√	Problem	Conditions To Check For	Conditions That Should Exist
M,S	Pipes		Sediment & debris	Accumulated sediment that exceeds 20% of the diameter of the pipe.	Pipe cleaned of all sediment and debris.
M			Vegetation	Vegetation that reduces free movement of water through pipes.	All vegetation removed so water flows freely through pipes.
A			Damaged (rusted, bent, or crushed)	Protective coating is damaged; rust is causing more than 50% deterioration to any part of pipe.	Pipe repaired or replaced.
M				Any dent that significantly impedes flow (i.e., decreases the cross section area of pipe by more than 20%).	Pipe repaired or replaced.
M				Pipe has major cracks or tears allowing groundwater leakage.	Pipe repaired or replaced.
M,S	Open Ditches		Trash & debris	Dumping of yard wastes such as grass clippings and branches into basin. Unsightly accumulation of nondegradable materials such as glass, plastic, metal, foam, and coated paper.	Remove trash and debris and dispose as prescribed by the County.
M			Sediment buildup	Accumulated sediment that exceeds 20% of the design depth.	Ditch cleaned of all sediment and debris so that it matches design.
A			Vegetation	Vegetation (e.g., weedy shrubs or saplings) that reduces free movements of water through ditches.	Water flows freely through ditches. Grassy vegetation should be left alone.
M			Erosion damage to slopes	Check around inlets and outlets for signs of erosion. Check berms for signs of sliding or settling. Action is needed where eroded damage over 2 inches deep and where there is potential for continues erosion.	Find caused of erosion and eliminated them. Then slopes should be stabilized by using appropriate erosion control measure(s); e.g., rock reinforcement, planting grass, compaction.

Maintenance Checklist for Conveyance Systems (Pipes, Ditches and Swales) - continued

A			Rock lining out of place or missing (if applicable)	Maintenance person can see native soil beneath the rock lining.	Replace rocks to design standard.
Varies	Catch basins			See Catch Basins Checklist.	See Catch Basins Checklist.
M,S	Swales		Trash & debris	See above for Ditches.	See above for Ditches.
M			Sediment buildup	See above for Ditches.	Vegetation may need to be replanted after cleaning.
M			Vegetation not growing or overgrown	Grass cover is sparse and seedy or areas are overgrown with woody vegetation.	Aerate soils and reseed and mulch bare areas. Maintain grass height at a minimum of 6 inches for best stormwater treatment. Remove woody growth, recontour, and reseed as necessary.
M,S			Erosion damage to slopes	See above for Ditches.	See above for Ditches.
M			Conversion by homeowner to incompatible use	Swale has been filled in or blocked by shed, woodpile, shrubbery, etc.	If possible, speak with homeowner and request that swale area be restored. Contact the County to report problem if not rectified voluntarily.
A			Swale does not drain	Water stands in swale or flow velocity is very slow. Stagnation occurs.	A survey may be needed to check grades. Grades need to be in 1-5% range if possible. If grade is less than 1% underdrains may need to be installed.
M,S			Trash or litter	Dumping of yard wastes such as grass clippings and branches onto grounds. Unsightly accumulation of nondegradable materials such as glass, plastic, metal, foam, and coated paper.	Remove trash and debris and dispose as prescribed by the County.
M,S			Erosion of Ground Surface	Noticeable rills are seen in landscaped areas.	Causes of erosion are identified and steps taken to slow down/spread out the water. Eroded areas are filled, contoured, and seeded.
A	Trees and shrubs		Damage	Limbs or parts of trees or shrubs that are split or broken which affect more than 25% of the total foliage of the tree or shrub.	Trim trees/shrubs to restore shape. Replace trees/shrubs with severe damage.
M				Trees or shrubs that have been blown down or knocked over.	Replant tree, inspecting for injury to stem or roots. Replace if severely damaged.
A				Trees or shrubs which are not adequately supported or are leaning over, causing exposure of the roots.	Place stakes and rubber-coated ties around young trees/shrubs for support.

If you are unsure whether a problem exists, please contact a Professional Engineer.

Comments:

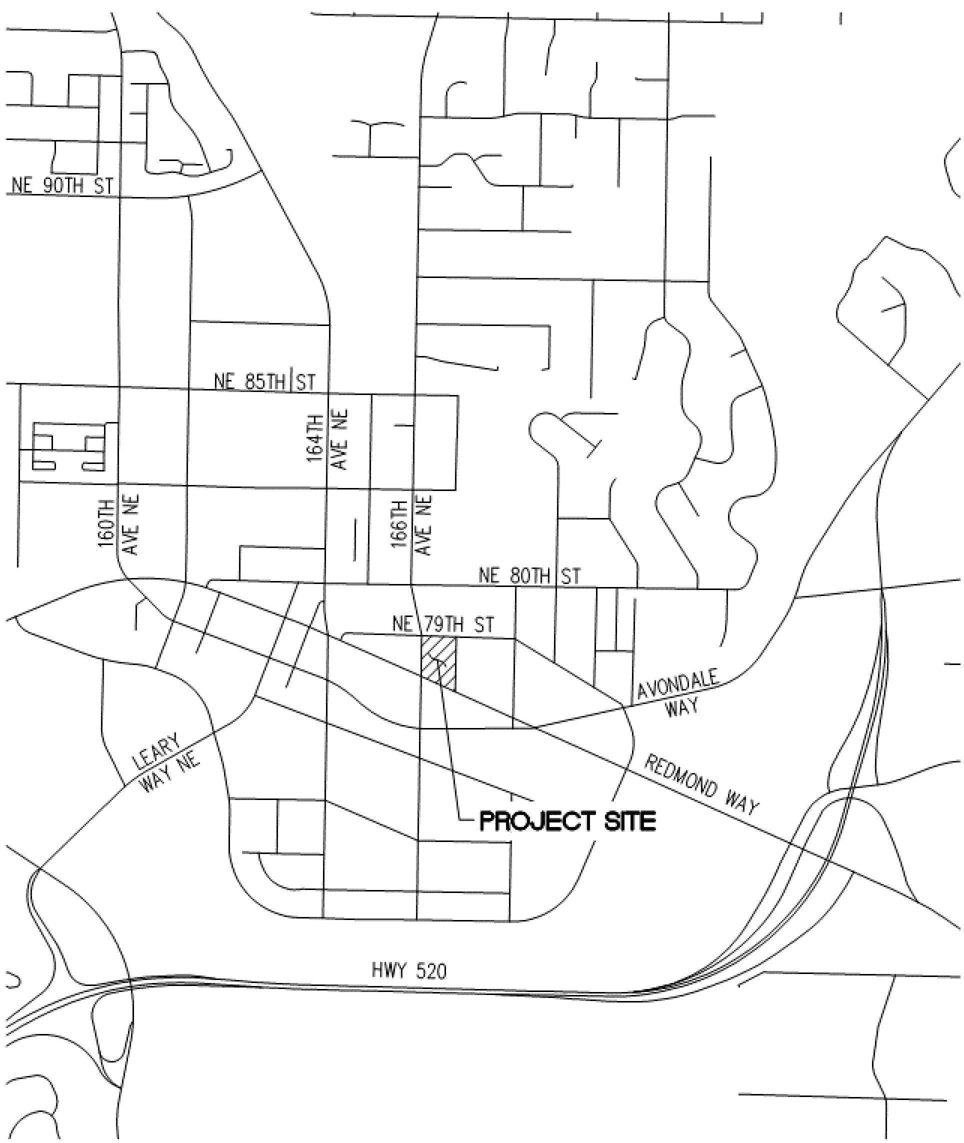
A = Annual (March or April, preferred)
M = Monthly (see schedule)
S = After major storms (use 1-inch in 24 hours as a guideline)

CONCLUSION

The proposed storm water management system for this project has been designed in accordance with regulatory criteria described above and consistent with sound engineering practice. This design has incorporated storm water detention and storm water quality best management practices. Therefore, no significant adverse impacts to the upstream or downstream storm water management systems are expected as a result of the proposed development.

APPENDIX A – FIGURES

SEC 12, TWN 25 N, RNG 5 E W.M.



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ANDERSON PARK HOTEL
NE CORNER OF REDMOND WAY AND 166TH AVE NE
REDMOND, WA
VICINITY MAP
1" = 1000'
2/3/2016
EX 01

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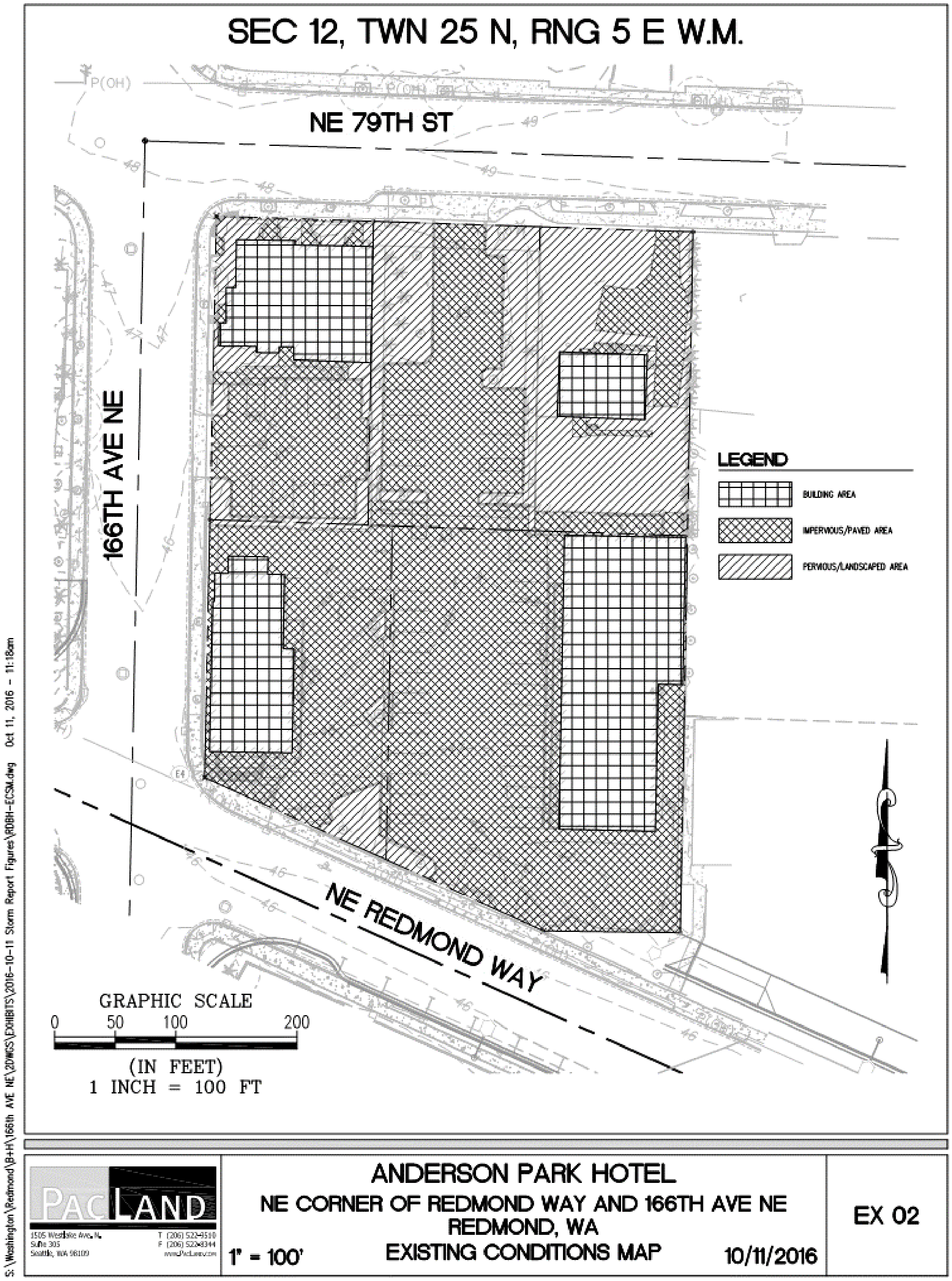
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NE 79TH ST
166TH AVE NE
NE REDMOND WAY
LEGEND
BUILDING AREA
IMPERVIOUS/PAVED AREA
PERVIOUS/LANDSCAPED AREA
GRAPHIC SCALE
0 50 100 200
(IN FEET)
1 INCH = 100 FT
PACLAND
ANDERSON PARK HOTEL
NE CORNER OF REDMOND WAY AND 166TH AVE NE
REDMOND, WA
1" = 100'
EXISTING CONDITIONS MAP
10/11/2016
EX 02
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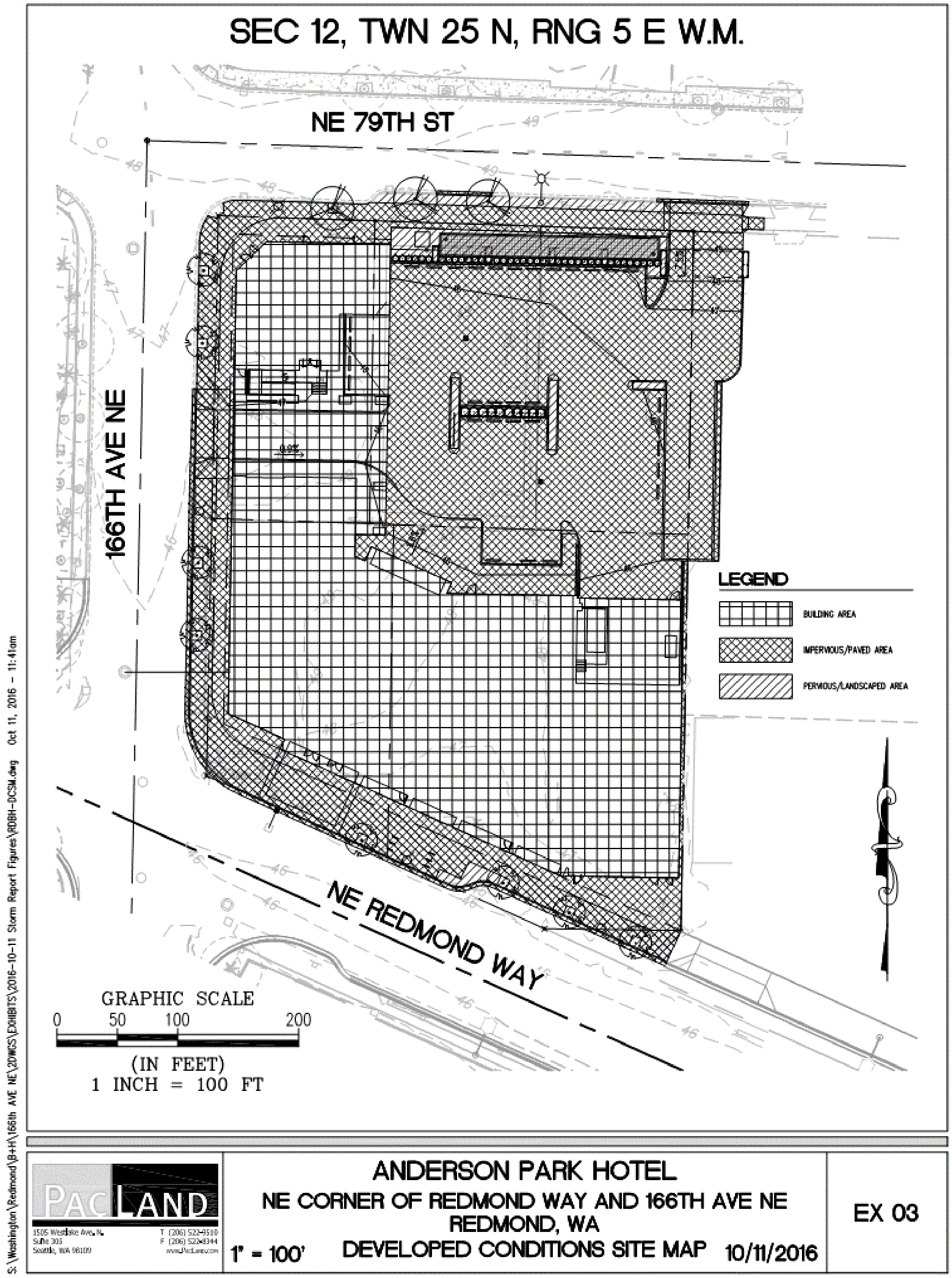
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ANDERSON PARK HOTEL
NE CORNER OF REDMOND WAY AND 166TH AVE NE
REDMOND, WA
1" = 100'
DEVELOPED CONDITIONS SITE MAP
10/11/2016
EX 03

APPENDIX B – PLANS

APPENDIX C – GEOTECHNICAL ENGINEERING REPORT

(not including appendices)

REPORT

DRAFT

GEOTECHNICAL REPORT

Anderson Park Hotel
7828 166th Avenue NE
16630 and 16648 NE Redmond Way
16651 NE 79th Street
Redmond, Washington

Submitted To: Washington Real Estate Holdings, LLC
600 University Street, #2820
Seattle, WA 98101

Submitted By: Golder Associates Inc.
18300 NE Union Hill Road, Suite 200
Redmond, WA 98052 USA

January 14, 2016

Project No. 1546103

Golder Associates

Table of Contents

List of Tables

List of Figures

List of Appendices

1.0 INTRODUCTION

1.1 Purpose of Report

This geotechnical report presents the results of the geotechnical investigation performed by Golder Associates Inc. (Golder) for the proposed Anderson Park Hotel project located at the northwest corner of the intersection of NE Redmond Way (State Route 202) and 166th Avenue NE in Redmond, Washington, as shown in Figure 1. The project site consists of five King County parcels as shown in Figure 2. This report also provides geotechnical recommendations for design and construction of the project.

Golder's scope of work was completed in accordance with the scope of our proposal dated December 1, 2015. It was based on the request for proposal (RFP) dated November 20, 2015 prepared by B + H Architects (B+H), subsequent conversations with B+H, review of existing geotechnical information, a site visit on November 30, 2015, and our own experience with similar projects in Redmond.

1.2 Project Description

The project site is located in downtown Redmond (Figure 1) and is comprised of five adjoining King County parcels (122505-9150, 122505-9154, 122505-9065, 122505-9103, and 122505-9213) covering about 1.2 acres (Figure 2). The site is bounded by NE 79th Street to the north, 166th Avenue NE to the west, Redmond Way (State Route 202) to the south, and two commercial properties to the east. The site is relatively level with a slight slope down towards the south. The surface elevation is approximately 50 feet above mean sea level (asml) (NAVD88 datum).

Site development currently includes mixed-use commercial, restaurants, retail, and a residence that is used for business purposes. Four buildings are located on each of the four corner parcels while the northern-central parcel is a vacant lot used as an asphalt-surfaced driveway and parking area (122505-9154). The remaining areas of the project site include asphalt-surfaced parking lots used by customers and employees.

The development plan for the property calls for demolition of all existing structures and construction of a Select Service Hotel to cover the majority of the project site. The hotel will contain 170 rooms, a restaurant/bar area, café, meeting rooms, indoor pool, fitness area, and hotel laundry. The building will be six stories tall with an adjoining 4-level parking structure. One level of the parking structure will be below-grade. The estimated deepest excavation depth is about 14 feet below existing grades, about elevation 36 feet (NAVD88).

2.0 SUBSURFACE INVESTIGATION

2.1 General

Golder's geotechnical subsurface investigation consisted of advancing four boreholes (GB-1, GB-2, GB-3, and GB-4). The boreholes were advanced to evaluate the soil and groundwater conditions underlying the project area. Approximate exploration locations are shown in Figure 2. Borehole records are presented in Appendix A. Three monitoring wells were installed on the site by others (B-1 through B-3) and are included in Figure 2 and discussed in Section 2.3 below.

The soil conditions encountered in the geotechnical borings were examined and classified in general accordance with Golder's Technical Procedures, which is summarized in the Method of Soil Classification in Appendix A. Pertinent information was recorded, including sample depths, stratigraphy, groundwater occurrence, and engineering characteristics.

The stratigraphic contacts shown on the borehole logs represent the approximate boundaries between soil units; actual transitions may be more gradual. The subsurface conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of conditions at other locations and times.

2.2 Borehole Explorations

Four geotechnical boreholes labeled GB-1 through GB-4 were drilled on December 9 and 10, 2015. All four boreholes were advanced using a B-59 Mobile truck-mounted drill rig operated by Holt Services, Inc. under the full-time supervision of Golder geologist, John Hennessy. The completed boreholes were backfilled with bentonite chips, in accordance with Washington State Department of Ecology, and patched at the surface with cold asphalt or grass sod, to match the pre-existing ground surface material. Samples were retained in sealed plastic jars and transported to our Redmond laboratory for further evaluation and laboratory testing. Drill cuttings were collected in steel drums and removed from the site. Borehole records are provided in Appendix A. A general summary of the boreholes advanced by Golder is listed in Table 2-1.

Table 2-1: Summary of Boreholes

Soil Borehole	Date Drilled	Drilling Method	Approximate Ground Surface Elevation (ft)	Borehole Depth (ft-bgs)
GB-1	12/09/2015	Mud Rotary	48	31.5
GB-2	12/09/2015	Mud Rotary	49	31.4
GB-3	12/10/2015	Mud Rotary	49	31.5
GB-4	12/10/2015	Hollow Stem Auger	50	31.5

Notes: ft = feet; ft-bgs = feet below ground surface

Drilling and sampling were performed in general accordance with Golder's Technical Procedures. Standard penetration test (SPT) samples were attempted at approximate 2.5-foot depth intervals to 15 feet and then at 2.5- to 5-foot intervals to the depths explored, depending on subsurface conditions. Samples were retrieved using 2-inch diameter split-spoon samplers advanced with a 140-pound automatic drop hammer falling a distance of 30 inches for each strike, in accordance American Society for Testing and Materials (ASTM) Standard D1586-11. At times, it was necessary to use 3-inch diameter split spoon samplers to attempt to retrieve an adequate amount of sample material for laboratory testing when coarse gravels were encountered. When the larger sampler was used it was noted on the boring record.

The number of hammer blows required to advance the samplers every 6 inches over three successive increments was recorded. The standard penetration resistance (N) of the soil is calculated as the sum of the number of blows required for the final 12 inches of sampler penetration. The N-value is an indication of the relative density of cohesionless soils and consistency of cohesive soils. If 50 blows are recorded for a single 6-inch interval, the test is terminated and the blow count is recorded as 50 blows for the total length of penetration. The maximum number of blows for a single 6-inch interval is increased to 100 when using the 3-inch diameter sampler and recorded N-values need to be corrected for proper evaluation.

2.3 Monitoring Wells

On May 11, 2015, Terracon Consultants, Inc. installed three monitoring wells, B-1 through B-3 on the southeastern parcel of the project site as shown in Figure 2. These wells were installed as part of an environmental subsurface investigation (Terracon 2015).

Groundwater level readings were collected by Terracon during installation and again about 24 hours after installation. Golder measured groundwater in these wells during our subsurface investigation as well and the data are summarized in Table 3-1. Copies of the borehole logs including monitoring installation records and construction are included in Appendix B.

2.4 Geotechnical Laboratory Testing

Laboratory testing was performed on selected samples from Golder borings to confirm visual soil classifications on the boring records and to provide information for liquefaction analysis and infiltration design parameters. The index testing included eight sieve analyses (ASTM D422) with moisture content determinations. The results of the laboratory tests are presented in Appendix C.

3.0 SUBSURFACE CONDITIONS

This section presents a discussion of site geology and subsurface conditions to support geotechnical recommendations.

3.1 Regional Geologic Setting

The Anderson Park Hotel project site lies in a broad alluvial valley occupied by the Sammamish River, Bear Creek, and Evans Creek. The valley is underlain at depth by fine grained deposits of laminated silt and clay referred to as transitional beds (Minard et al. 1988). Overlying the transitional beds are alluvial deposits consisting of sand and gravel deposited after the retreat of the Vashon Stade of the Fraser Glaciation. The alluvium deposits form a productive municipal drinking water aquifer for the City.

According to the City of Redmond's Wellhead Protection Plan and borehole data obtained by Golder from nearby properties, two geologic units are present at shallow depths below the site. The uppermost unit is a clean sand and gravel (Alluvium and Recessional Outwash) that is about 35 to 70 feet thick in the downtown Redmond area (Parametrix 1997, Golder 2003a, Golder 2003b), with the base of the deposits at or below sea level. The alluvium/outwash is underlain by fine-grained silts and sands (transitional beds). The geologic units encountered during Golder's site investigation are described below.

3.2 Observed Soil Units

Geologic units encountered during the borehole exploration have been interpreted to include fill and alluvium, in agreement with the geologic map (Minard et al. 1988). General descriptions of these units are presented below. Specific soil descriptions are provided in the borehole exploration records in Appendix A.

- **Fill:** Fill or modified land refers to soil placed or modified by human activity. Fill was encountered in all four boreholes at the ground surface extending about 2 to 4.5 feet below ground surface (bgs). The fill generally consisted of asphalt over compact sand and gravel with cobbles.
- **Alluvium:** Alluvium was encountered beneath the fill in all four of the boreholes and extended to the depths explored, about 31.5 feet bgs. The alluvium was generally compact to dense sandy gravel, gravelly sand, sand, and gravel.

3.3 Groundwater Conditions

Groundwater was encountered in all four Golder geotechnical boreholes. However, no monitoring wells were installed. Groundwater level was measured in the three existing monitoring wells B-1, B-2, and B-3. All groundwater measurements are summarized in Table 3-1. The groundwater elevations should be considered approximate since a survey of the wells and borings has not been completed.

The seasonal groundwater elevation fluctuation in the vicinity of the Anderson Park Hotel project site can range from a low of about 26 feet amsl to a high of about 32.5 feet amsl based on groundwater elevations reported by the City of Redmond for groundwater monitoring well MW008 between February 2008 and January 2009 and between March 2014 and December 2014. MW008 is located on the northwest corner of NE 79th Street and 166th Avenue NE, near the northwest corner of the project site. Longer-term (February 2008 to May 2014) groundwater elevation data are available from another City well, MW009, located about 480 feet east of MW009. The groundwater elevation in MW009 has ranged from about 26 to 32 feet amsl, with the exception of higher elevations of 33.5 and 34 feet amsl for short periods in January 2009 and December 2010, and a lower elevation of about 24 feet amsl in September 2013.

Table 3-1: Anderson Park Hotel Groundwater Monitoring

Exploration Number	Approximate Ground Surface Elevation	Date	Depth to Groundwater (feet bgs)	Approximate Groundwater Elevation (feet, NAVD88)	Comments
MW008[1]	48	4/15/2015	20.5	27.5	City of Redmond well
B-1	49	5/11/2015	20	29	Measured by Terracon at time of drilling
		5/12/2015	18	31	Measured by Terracon day after drilling
		12/9/2015	15.75	33.25	Measured by Golder
		12/10/2015	15.0	34	Measured by Golder
B-2	50	5/11/2015	20	30	Measured by Terracon at time of drilling
		5/12/2015	18.75	31.25	Measured by Terracon day after drilling
		12/4/2015	17.9	32.1	Measured by Golder
		12/9/2015	16.65	33.35	Measured by Golder
		12/10/2015	16.05	33.95	Measured by Golder
B-3	49	5/11/2015	17.5	31.5	Measured by Terracon at time of drilling
		5/12/2015	17.9	31.1	Measured by Terracon day after drilling
		12/4/2015	16.85	32.15	Measured by Golder
		12/9/2015	15.65	33.35	Measured by Golder
		12/10/2015	14.95	34.05	Measured by Golder
GB-1	48	12/9/2015	16	32	Groundwater depth approximate at time of drilling[3]
GB-2	49	12/9/2015	15.5	33.5	Groundwater depth approximate at time of drilling[3]

Exploration Number	Approximate Ground Surface Elevation	Date	Depth to Groundwater (feet bgs)	Approximate Groundwater Elevation (feet, NAVD88)	Comments
GB-3	49	12/10/2015	16	32	Groundwater depth approximate at time of drilling[3]
GB-4	50	12/10/2015	18.5	31.5	Depth measured at time of drilling[4]

Notes:
1) MW008 is a previously installed City of Redmond well located at the northwest adjacent property to the Anderson project site; copy included in Appendix D.
2) Pre-existing monitoring wells at the project site as installed by Terracon Consultants, Inc. (Terracon) on May 11, 2015. Terracon's borehole and monitoring well installation records provided in Appendix B.
3) Boreholes advanced using mud rotary drilling methods and downhole groundwater levels could not be directly measured due to use of drilling fluid; groundwater depths listed were approximated at time of drilling based on drilling observations and current groundwater levels measured in nearby Terracon monitoring wells.
4) Borehole advanced using hollow stem auger method and the downhole depth to groundwater was measured at time of drilling.
5) NAVD88 datum.

4.0 DESIGN RECOMMENDATIONS

The following sections present recommendations for the design of the proposed structures.

4.1 Seismic Design

The 2012 International Building Code (IBC 2012) seismic design section provides information to be used as the basis for seismic design of structures.

4.1.1 Site Class

Section 1613 of the 2012 IBC provides information on earthquake loads and site class. Section 1613.3.2 of 2012 IBC states "[b]ased on the site soil properties, the site shall be classified as *Site Class* A, B, C, D, E, or F in accordance with Chapter 20 of ASCE 7." The site class can be classified according to the average soil profile properties in the first 100 feet bgs. Based on the SPT N-values recorded in the boreholes, it is our opinion that the site should be classified as Site Class D.

4.1.2 Ground Motion Parameters

Ground motion parameters used for design per the 2012 IBC include the site coefficient and mapped spectral accelerations, which can be found in section 1613.3. The mapped spectral accelerations correspond to Class B conditions. Accordingly, the spectral response accelerations should be adjusted for the site-specific Class D soil conditions.

The following design parameters are based on the IBC Maximum Considered Earthquake (MCE) Ground Motion, the 0.2-second spectral acceleration (Ss), and the 1.0-second spectral acceleration (S1) for the project site. The interpolated probabilistic ground motion values in percent gravity were obtained from the United States Geological Survey (USGS) US Seismic Design Maps (USGS 2015). The following results were obtained for latitude 47.673416 and longitude -122.118357 (a point located near the center of the site):

- Short (0.2 second) Spectral Response
  - Ss: 1.254 g
  - Sms: 1.254 g
  - Sds: 0.836 g
- Long (1.0 second) Spectral Response
  - S1: 0.481 g
  - Sm1: 0.730 g
  - Sd1: 0.487 g

4.1.3 Liquefaction Assessment

Loose to compact soil deposits below the water table can be susceptible to a phenomenon called liquefaction during earthquake shaking. During liquefaction the soil temporarily loses strength and acts like a viscous fluid. Liquefaction can cause ground settlement potentially affecting building foundations as well as floating of buried objects such as tanks and pipes. A liquefaction assessment is typically carried out for buildings that will be supported on granular soil deposits below the water table such as the Anderson Hotel. The assessment looks at the density and particle size distribution of the soil as well as the design earthquake event for the project area. A liquefaction assessment was completed using the information from the four Golder boreholes completed for this report. Details of the liquefaction assessment are discussed below.

4.1.3.1 Assumptions

The peak ground acceleration (PGA) values on bedrock for seismic design were estimated using the USGS US Seismic Design Maps (USGS 2015). Assuming a risk level of 2% probability of exceedance (PE) in 50 years (approximately a 2,500-year recurrence interval) for the site, a PGA of 0.51g can be used for liquefaction assessment. An earthquake of Magnitude 6.8 was assumed for analysis purposes.

The groundwater was assumed at a maximum elevation of 34 feet, consistent with highest elevation measured in nearby City of Redmond monitoring wells.

4.1.3.2 Methodology

The liquefaction potential of the soil was evaluated using commercially available computer program LiquefyPro version 5.8a, a proprietary software code produced by CivilTech Software of Seattle, Washington (LiquefyPro 2009).

LiquefyPro uses the procedure presented by Youd and Idriss (2001) to assess the liquefaction hazard of the soil. In this procedure, the cyclic shear stress induced by the earthquake is compared with the cyclic resistance of the soil. If the induced shear stress is greater than the resistance of the soil, liquefaction is likely to occur.

The earthquake-induced cyclic shear stress was calculated using the simplified procedure of Seed and Idriss (1971) using the estimated peak horizontal ground acceleration. The cyclic stress ratio (CSR) is a function of the total vertical overburden stress, the effective vertical overburden stress, the peak horizontal ground acceleration, and a stress reduction coefficient.

The liquefaction or cyclic resistance of the soil was calculated using the procedure in Youd and Idriss (2001) for in-situ test data from the SPT tests. The SPT blow counts (N) are corrected for the vertical effective stress (N_1), hammer efficiency $(N_1)_{60}$, rod lengths, fines content of the soil, and sampler size.

The corrected value is the $(N_1)_{60cs}$, which is correlated to the cyclic resistance ratio of the soil (CRR). The CRR is adjusted for the earthquake magnitude.

4.1.3.3 Liquefaction Results and Discussion

The results of the liquefaction assessment indicate that liquefaction induced by the 2,500-year design event is not likely to occur in the soils observed below the water level in the four onsite boreholes. Therefore, it is our opinion that the risk of liquefaction on the site is low.

4.1.4 Seismic Surcharge on Walls

A seismic surcharge should be added to the earth pressures on below grade basement walls. Recommendations for the seismic surcharge are included in Section 4.5 "Permanent Wall Design Criteria" of this report.

4.2 Groundwater

Groundwater levels in the alluvial aquifer beneath the site have been measured in a network of monitoring wells installed by the City. The seasonal groundwater elevation fluctuation in the vicinity of the site ranges from a low of about 26 feet amsl to a high of about 32.5 feet amsl based on groundwater elevations measured by the City in MW008 located on the corner of NE 79th Street and 166th Avenue NE, near the northwest corner of the site.

We understand the planned parking garage and building foundations will be no deeper than approximately elevation 36 feet based on discussions with the architect and structural engineer. This is about 2 feet higher than the maximum measured seasonal groundwater elevation of approximately elevation 34 feet. We recommend that a seasonal high groundwater elevation of 34 feet be planned for in the building design.

4.3 Foundations

The foundations for the proposed buildings may consist of shallow isolated and continuous spread footings bearing on the native alluvium soil.

Fill was encountered to a depth of 2 to 4.5 feet bgs in the boreholes. If uncontrolled fill is encountered at the footing elevation during construction, the uncontrolled fill should be removed and replaced with structural fill in accordance with recommendations contained in Section 5.4.

4.3.1 Spread Footings

Conventional shallow isolated or continuous foundations placed on compact to very dense native soil or compacted structural fill should be designed based on the following recommendations. Refer to Section 5.0 for construction considerations pertaining to foundations.

Foundation recommendations are based on the current project description as described in Section 1.0. If the configuration of the proposed building changes, Golder should be notified to review the updated plans and revise the foundation recommendations accordingly. In particular, the allowable bearing capacity will change if different footing sizes and embedment depths are used for design.

- Design isolated footings using a maximum allowable bearing pressure of 4 kips per square foot (ksf) assuming a minimum footing width of 2 feet, a maximum footing width of 8 feet, maximum estimated settlement on the order of 1 inch, and differential settlement on the order of ½ inch.
- Design continuous footings using a maximum allowable bearing pressure of 3 ksf assuming a minimum footing width of 2 feet and a maximum footing width of 4 feet.
- The maximum allowable bearing pressures meet the required factor of safety according to IBC.
- The recommended maximum allowable bearing pressures are gross bearing pressures.
- The recommended maximum allowable bearing pressures are expected to result in less than 1 inch of total settlement.
- The values presented may be increased by one-third for short-term wind and seismic loading.
- Isolated and continuous footings should be embedded at least 18 inches below the adjacent finished grade.
- The above recommendations are based on concentric pressures applied at the base of the footings. In the case of eccentric pressures (e.g., due to lateral loads), Golder may need to re-evaluate the recommended pressures.

A representative from Golder should observe the foundation bearing soils prior to placement of forms and rebar to verify the foundation bearing soils are consistent with the soils encountered at the time of this study.

Building foundations must resist lateral loads due to earth pressures, wind, and seismic events. For design purposes, these loads can be resisted simultaneously by:

- BASE FRICTION: An allowable value of 0.4 can be assumed for base friction between the soil and spread footings. This value includes a factor of safety of 1.5. The allowable base friction value may be increased by one-third for the seismic loading.
- PASSIVE RESISTANCE ON BASEMENT WALLS FOUNDED AGAINST SOIL: We recommend that the allowable passive pressure be based on a fluid with a density of 270 pounds per cubic foot (pcf) (including a factor of safety of 1.5). This value can be increased by one-third for seismic loads.
- PASSIVE RESISTANCE ON SIDES OF SHALLOW FOOTINGS: For design purposes, we recommend that the allowable passive pressure be based on a fluid with a density of 270 pcf (including a factor of safety of 1.5) for shallow foundations. The allowable passive resistance can be increased by one-third for seismic loading. Since some disturbance is likely to occur during construction, we recommend the upper 1 foot of passive resistance be neglected.

Please refer to the drainage provision section (Section 4.6) of this report for foundation drainage criteria.

4.4 Slab Subgrade

Conventional slab-on-grade floors can be supported on a subgrade of the native bearing soils or on structural fill placed and compacted as noted in the Earthworks section of this report (Section 5.5). Slab-on-grade floors should not be founded on organic soils, loose soils, or uncontrolled fills.

We recommend that slabs be underlain by a capillary break material, consisting of a minimum thickness of 4 inches of clean, free draining gravel, or crushed rock containing less than 2% fines passing the US No. 200 sieve (based on the minus US No. 4 sieve fraction) meeting the following specification:

Table 4-1: Capillary Break Gradation

Sieve Size or diameter (in)	Percent Passing
1 inch	100 %
No. 4	0% – 70%
No. 10	0 – 30%
No. 100	0 – 5%
No. 200	0 – 2 %

Vapor transmission through floor slabs is an important consideration in the performance of floor coverings and controlling moisture in structures. For storage, possible moisture effects on materials placed on bare concrete floors should also be considered. The identification of alternatives to prevent vapor transmission is outside of our expertise. A qualified architect or building envelope consultant can make recommendations for reducing vapor transmission through the slab, based on the building use and flooring specifications.

4.5 Permanent Wall Design Criteria

The design lateral pressure on permanent basement walls depends on the depth below the groundwater table. The preliminary design ground water table elevation is 34 feet. We understand the planned below grade parking level will be above the seasonal high water table elevation. Recommended lateral earth pressure coefficients for design of permanent walls are shown in Table 4-2. The earth pressure coefficient can be used in combination with the triangular pressure distribution shown in Figure 3. Where typical passenger vehicle traffic loads will occur adjacent to the wall, a uniform vertical surcharge load of 100 pounds per square foot (psf) should be added. Additional surcharges due to adjacent building or foundation should be added to the design pressures as required. For estimation of surcharge loads on walls, refer to Figure 4.

The lateral earth pressure coefficients shown in Table 4-2 can also be used for design of permanent backfilled walls provided that the backfill meets the gradation for "Gravel Backfill for Walls" - WSDOT Standard Specification 9-03.12(2) (WSDOT 2014). The onsite native clean sand and gravels are also suitable for backfill behind permanent walls design for the lateral earth pressure coefficients shown in Table 4-2.

The seismic coefficient used to calculate the seismic earth pressure coefficient is taken as a portion of the PGA value adjusted for site effects. If permanent deflection of a few inches following a seismic event is acceptable, then the PGA value can be reduced prior to calculating seismic earth pressures. Otherwise, the full value of the PGA should be used for the seismic coefficient. We have provided seismic active earth pressure coefficients for seismic coefficients equal to the full PGA value and a reduced PGA value in Table 4-2.

If the permanent wall is designed for at-rest earth pressures, it should be noted that the seismic earth pressure coefficients (Kae) in Table 4-2 are technically seismic active earth pressure coefficients, not seismic at-rest earth pressure coefficients. The values of Kae in Table 4-2 are based on the Mononobe-Okabe (M-O) Method which has been found to provide reasonable seismic earth pressure for basement walls and cross-braced excavations (Sitar et. al. 2012). The values of Kae can be used in place of Ka in the pressure diagram in Figure 3 for seismic design.

Permanent walls shall have drainage provisions, as discussed in Section 4.6, to provide full wall drainage above elevation 34 feet.

Table 4-2: Design Parameters for Permanent Wall Design

Design Parameter	Value
Active Earth Pressure Coefficient (Ka)	0.31
At-Rest Earth Pressure Coefficient (K_0)	0.47
Seismic Active Earth Pressure Coefficient (Kae1)	0.65
Seismic Active Earth Pressure Coefficient (Kae2)	0.49
Passive Earth Pressure Coefficient (Kp)	3.25
Total Unit Weight ($\gamma 1$)	125 pounds per cubic foot

Notes:

1. Use Ka for the design of permanent cantilever walls free to rotate about the top.
2. Use K_0 for the design of permanent basement walls restrained at the top.
3. Use Kae1 for the design of permanent walls that cannot deflect during design earthquake.
4. Use Kae2 for the design of permanent walls where permanently deflections of 1 to 2 inches resulting from the design earthquake are acceptable.
5. Value for passive earth pressure coefficient (Kp) is un-factored. Apply a factor of safety = 2.0 for IBC allowable stress load combinations. Use Kpe = 2.8 for seismic design.
6. Use Total Unit Weight above elevation 34 feet.

4.6 Permanent Drainage Provisions

We understand the planned below grade parking level will be above the seasonal high water table elevation of 34 feet. For all structures designed above elevation 34 feet, we recommend that the following measures to provide drainage:

- WALL DRAINS: Drainage behind backfilled walls can consist of a full face geocomposite drainage mat or a minimum of a two foot wide zone of clean sand and gravel fill with less than 5% passing the No. 200 sieve. For shoring, temporary drainage typically consists of a grid of geocomposite drainage strips with the permanent drainage consisting of full face geocomposite drainage mat that that is tied into an interior perimeter footing drain.
- FOOTING DRAIN: A perimeter footing drain should also be placed consisting of a 4 inch diameter heavy-walled perforated PVC pipe or equivalent. The pipe should be surrounded by at least 12 inches of drainage material. Cleanouts should be provided. The drain should flow by gravity to the storm drain system.

We recommend that Golder review the final plans in regards to the need for subsurface permanent drainage provisions.

4.7 Waterproofing

A building envelope consultant should be retained to provide waterproofing recommendations. Generally, waterproof barriers should be used between buried walls and the earth, and for walls cast directly against the shoring. For areas that are shored, the waterproofing can be placed over a geocomposite drain prior to pouring or shooting the concrete wall. A structural building envelope consultant or an architect can make recommendations regarding waterproofing design specifications.

4.8 Temporary Shoring Design Criteria

4.8.1 Soldier Pile and Tieback Design Criteria

A cantilever soldier pile shoring system or a soldier pile and tieback shoring system with one row of anchors appears appropriate for supporting the proposed excavation depths. The design earth pressure diagram is shown in Figure 3 for the active condition of cantilever soldier pile walls and for soldier pile walls with one row of anchors. If deformations of the shoring wall must be limited, active earth pressure coefficient can be replaced with at-rest earth pressure coefficient for the design of the shoring walls. Earth pressure coefficients and unit weights for soils are included in Table 4-3. The earth pressure recommendations are based on the current project description described in Section 1.0. If the configuration of the proposed buildings changes, Golder should be notified to review the updated plans and revise earth pressure recommendations accordingly.

Table 4-3: Design Parameters for Temporary Shoring Design

Design Parameter	Value
Active Earth Pressure Coefficient (Ka)	0.31
Passive Earth Pressure Coefficient (Kp)	3.25
Total Unit Weight ($\gamma 1$)	125 pounds per cubic foot
Submerged Unit Weight ($\gamma 2$)	63 pounds per cubic foot

Notes:
1. Value for passive earth pressure coefficient (Kp) is un-factored.
2. Use Total Unit Weight above elevation 34 feet.

Additional lateral surcharges should be added to the design earth pressures to account for any vertical surcharges adjacent to the excavation, such as the tower footings, surrounding buildings, traffic surcharges, and construction surcharge loadings, including those from mobile cranes and pump trucks. Surcharges on shoring walls can be calculated using the appropriate equation presented in Figure 4. The earth pressures presented assume level ground above the top of the shoring. If sloping ground is present, a uniform horizontal surcharge equal to one-half of the height of the slope multiplied by the unit weight of the soil and the appropriate earth pressure coefficient should be added to the lateral earth pressure for level ground.

The embedment depth of soldier piles below the base of the excavation should be designed to provide force and moment equilibrium. Soldier piles should be embedded a minimum 10 feet below the base of the excavation.

For vertical structural loads on soldier piles spaced at least 2.5 pile diameters center to center, the following design criteria is recommended:

- Minimum embedment of 10 feet below the base of the excavation
- Allowable end-bearing resistance of 20 ksf for piles end bearing in the dense alluvium
- Allowable side friction of 1 ksf below the base of the excavation

The soldier piles should be designed to have adequate vertical capacity to resist the vertical components of the tieback loads and also permanent structural loads, if required. Vertical capacity may be provided by a combination of end-bearing and friction below the base of the excavation.

It should be noted for design and planning purposes, the City of Redmond "Right-of-Way Management Temporary Shoring Requirements" do not allow for the installation of soldier piles or the backfill surrounding the soldier pile within or extending into the City Right-of-Way or Utility Easements.

4.8.1.1 Lagging

Lagging will be necessary to prevent caving of the soil face between the soldier piles. Lagging may be designed for 50% of the lateral soil pressures. However, for an 8-foot center to center span, a maximum thickness of 4 inches is recommended for No. 2 or better Hem-Fir wood lagging, even if the structural calculations show thicker wood lagging is required. Any voids behind the lagging should be backfilled with a permeable granular soil material that does not allow the buildup of hydrostatic pressure or controlled density fill (CDF). The excavation height prior to lagging installation should not exceed 4 feet, or less as required to maintain cut-face stability.

It should be noted for design and planning purposes, the City of Redmond "Right-of-Way Management Temporary Shoring Requirements" do not allow for the installation of lagging within or extending into the City Right-of-Way or Utility Easements.

4.8.1.2 Tieback Anchors

The anchor portion of the tieback should be located sufficiently far behind the excavation shoring to stabilize the excavation face. The no "load" zone limits is the area behind the soldier pile equal to a lateral distance from the base of the excavation equal to the exposed wall height (H) divided by four and a line sloping up and back at 60 degrees from horizontal.

The selection of tieback materials and installation methods should be the responsibility of the contractor. The actual adhesion values will depend on the materials and installation method and should be confirmed by testing.

For non-pressured grouted anchors, the allowable design concrete/soil friction value of 2 ksf (including a factor of safety of 2) in the dense alluvium can be used for preliminary design and cost estimating purposes and should be confirmed by testing prior to construction. For pressure grouted anchors, this value can typically be increased by two to three times.

A minimum anchor spacing of 6-foot center to center is recommended. Anchor holes should be drilled at an angle of 15 to 30 degrees down from horizontal. A minimum anchor bond length of 10 feet is recommended. The location and presence of existing features, such as utilities and foundations, should be checked during the design as these may affect the location and length of tieback anchors.

It should be noted for design and planning purposes, the City of Redmond "Right-of-Way Management Temporary Shoring Requirements" only allow for tiebacks to be installed within the City Right-of-Way or Utility Easements if certain guidelines are met. These guidelines include, among others, a minimum anchor depth of 8 feet bgs at the right-of-way or utility easement line and a minimum 5 feet of clearance below utilities.

4.9 Infiltration

Eight soil samples (three from GB-1, two from GB-2, one from GB-3, and two samples from GB-4) were submitted for grain size analyses in accordance with American Society for Testing and Materials (ASTM) Standards. The samples cover a range of elevations, from approximately 5 to 22.5 feet bgs. The saturated hydraulic conductivity of the materials was estimated from the grain size distribution using methods descried by Massman and others (2003). Table 4-4 summarizes the results of the analyses of the saturated hydraulic conductivity.

Table 4-4: Saturated Hydraulic Conductivity Results

Boring #	Sample #	Depth (feet bgs) Elevation (ft)	USCS	d_{10} (mm)	d_{60} (mm)	d_{90} (mm)	Fines (weight fraction)	Saturated Hydraulic Conductivity (in/hr)
GB-1	S-2	5	GW	0.32	8.44	18	0.049	96
GB-1	S-4	10	GW	1.36	7.41	18	0.022	9928
GB-1	S-6	15	SW-SM	0.14	1.42	27	0.073	23
GB-2	S-3	7.5	SP/SM GP/GM	0.18	4.32	18	0.067	41
GB-2	S-4	10	GW/GM SW/SM	0.11	7.12	17	0.083	32
GB-3	S-4	10	GW/GM SW/SM	0.23	8.18	20	0.056	58
GB-4	S-5	12.5	SP/GP	0.26	5.02	26	0.047	52
GB-4	S-9	22.5	SP	0.26	1.74	12	0.038	74

Notes:
bgs = below ground surface
USCS = Unified Soils Classification System
mm = millimeters
in/hr = inches per hour

Design infiltration rates were calculated by applying correction factors described in the Washington Department of Ecology Stormwater Management Manual for Western Washington (2014) to account for:

- Site variability and number of samples (CF_v) – a factor of 0.8 was applied.
- Test method (CF_t) – a factor of 0.4 is specified.
- Degree of influent control to prevent siltation and bio-buildup (CF_m) – a factor of 0.9 is specified.

The total correction factor was 0.288. The calculated design infiltration rates are summarized in Table 4-5. The calculated design infiltration rates range from 7 to 28 inches per hour (in/hr), with one gravel sample at 2,859 in/hr.

Table 4-5: Calculated Design Infiltration Rates

Borehole	Sample Number	Depth (feet bgs)	USCS	Saturated Hydraulic Conductivity (in/hr)	CF_v	CF_t	CF_m	Design Infiltration Rate (in/hr)
GB-1	S-2	5	GW	96	0.8	0.4	0.9	28
GB-1	S-4	10	GW	9928	0.8	0.4	0.9	2859
GB-1	S-6	15	SW-SM	23	0.8	0.4	0.9	7
GB-2	S-3	7.5	SP/SM GP/GM	41	0.8	0.4	0.9	12
GB-2	S-4	10	GW/GM SW/SM	32	0.8	0.4	0.9	9
GB-3	S-4	10	GW/GM SW/SM	58	0.8	0.4	0.9	17
GB-4	S-5	12.5	SP/GP	52	0.8	0.4	0.9	15
GB-4	S-9	22.5	SP	74	0.8	0.4	0.9	21

Notes:
bgs = below ground surface
USCS = Unified Soils Classification System
in/hr = inches per hour

4.9.1 Discussion and Design Considerations

The design infiltration rates calculated in Table 4-5 are variable, as is expected in a shallow alluvial soil. The high infiltration rate calculated from sample S-4 in boring GB-1 appears to be influenced by a localized clean gravel zone that may be limited laterally and vertically. Therefore, for long term design of we recommend the civil engineer use a design infiltration rate of 7 in/hr and follow the design recommendations listed below.

- The infiltration facilities should include an overflow to allow discharge to the City of Redmond stormwater system in the event the infiltration capacity is exceeded during large storm events.
- The groundwater elevation at the site is expected to range between about 26 to 31 feet amsl (NAVD88 datum) or about 17 to 22 feet below ground, based on historical groundwater level measurements in MW008 located at the corner of 166th Avenue NE and NE 79th Street, on the northeast side of the street, and the site elevation of about 49 feet. Based on the expected seasonal high groundwater level of about 14 feet below ground (elevation 34 feet (NAVD88)), the base of any stormwater infiltration facility should be no greater than 9 feet below ground in order to maintain a 5-foot separation from the seasonal high water table.
- No impermeable layers were observed during exploration drilling to the depths explored. If low permeability layers are observed during construction the recommendations in this report should be re-evaluated and modified if necessary.
- It is our understanding that the setback for the infiltration facility from the property line has been reduced. It is our professional opinion that this reduced setback distance will not result in negative impacts on the adjoining properties from the proposed drywells.

5.0 CONSTRUCTION RECOMMENDATIONS

Geotechnical related site construction would consist of demolition of onsite structures and utilities, shoring installation, excavation, subgrade preparation, placement of foundations, and placement and compaction of structural fill.

5.1 Erosion Control and Aquifer Protection

Erosion control and groundwater aquifer protection measures will need to be followed by the contractor during site construction. The project lies within Wellhead Protection Zone 1 and performance standards will apply to all activities at the site in accordance with Redmond Zoning Code (RZC 2011) 21.64.50. Best Management Practices (BMPs) in the civil design drawings should incorporate required aquifer protection measures as well as erosion control measures such as:

- Limit exposed cut slopes.
- Route surface water through temporary drainage channels around and away from exposed slopes.
- Use silt fences, straw, and temporary sedimentation ponds to collect and hold eroded material on the site.
- Seeding or planting vegetation on exposed areas where work is completed and no buildings are proposed.
- Retaining existing vegetation to the greatest possible extent.

We recommend that the contractor sequence excavations so as to provide constant positive surface drainage for rainwater and any groundwater seepage that may be encountered. This will require grading slopes, and constructing temporary ditches, sumps, and/or berms.

5.2 Temporary Construction Dewatering

Temporary construction dewatering is common for mixed use buildings in downtown Redmond where parking garages are typically about 1.5 levels below grade. The Anderson Park Hotel has only one level of below grade parking and based on historical groundwater elevation measurements in the project area it appears that foundations can be completed above the seasonal high groundwater level. Groundwater levels fluctuate with seasonal high levels typically occurring between November and March and seasonal low elevation in September or October. The deepest foundation excavations in downtown Redmond are typically planned for the late summer months to minimize the risk of encountering groundwater. There are also advantages to installing soldier pile shoring when groundwater levels are low to minimize caving in pile holes. Based on the information provided on the Anderson building foundation it is unlikely that construction dewatering will be needed to complete the foundation. Regardless the contractor must implement necessary dewatering, drainage, and surface water diversion measures to protect the excavation cut face and to prevent degradation of the excavation area and foundation subgrade during construction.

5.3 Temporary Shoring Installation and Testing

5.3.1 Soldier Pile Installation

The contractor should be required to prevent caving and loss of ground in all soldier pile excavations. Appropriate methods may be required to minimize caving and sloughing, such as drilling with slurry or the use of casing, to keep the soldier pile holes open. If slurry drilling is used or more than 1 foot of water is present in the bottom of the hole, placement of concrete by tremie methods will be required.

5.3.2 Tieback Anchor Installation

The contractor should be prepared to case the tieback holes if caving is encountered. In addition, occasional boulders should be anticipated. The tieback grout should be pumped into the anchor zone by tremie methods in order to force grout up from the bottom of the hole and to provide a continuous grouted anchor in the bonded zone. Tiebacks should be backfilled within the no load zone with a non-structural material, such as a mixture of sand slurry. A bond breaker should be applied to the anchor tendon through the no load zone. As an alternative, the no load zone can be backfilled with grout following anchor testing and lock off. Obstructions consisting of boulders, cobbles, and possible debris in the upper fill soils, and boulders and cobbles within the glacial soils should be anticipated and appropriate drilling methods should be used to allow for advance past the obstructions.

5.3.3 Tieback Anchor Testing and Lock Off

Verification testing is recommended on representative sacrificial tiebacks, and all production anchors should be proof tested to confirm the design capacity.

A minimum of two sacrificial 200% verification tests should be accomplished in each soil type used for anchorage to confirm the ultimate/soil friction capacity and the load/deformation performance of the tieback. Verification testing should be accomplished as soon as each soil type providing anchorage is encountered and prior to installation of production anchors. The location of the verification tests should be selected by the contractor and approved by the engineer. The drilling method, anchor diameter, grouting method, and depth of anchorage for the test anchors should be identical to the production anchors. The following procedures are recommended for verification testing:

1. The maximum test load should be 200% of the allowable concrete/soil adhesion capacity used for design. Measurement of the load should be accurate to 5 kips and movement to 0.001 inch.
2. The anchor should be loaded in increments of 25% of the 200% load, and each load held at least 10 minutes with measurement obtained at each increment. A creep test shall be performed at the 150% load increment. Anchor movement during the creep test shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes.
3. The total rate of movement during the creep test should be less than 0.08 inches of movement between 6 and 60 minutes and the creep rate is linear or decreasing.

4. Total movement at the maximum test load should exceed 80% of the theoretical elastic elongation of the unbonded length.

All production anchors should be proof tested using the following procedures:

1. The maximum test load is 130% of the design tieback load. Measurements of movement should be accurate to 0.001 inch and loads to 5 kips.
2. The anchor should be loaded in increments of 25% of design loads up to the 130% load. Each incremental load should be held long enough to obtain a stable displacement measurement. The 130% load should be held for 10 minutes. Anchor movement should be recorded at 1 minute, 2, 3, 5, 6, and 10 minutes. If the anchor movement between 1 and 10 minutes exceeds 0.04 inches, the maximum load test load shall be held an additional 50 minutes. If the load hold is extended, the anchor shall be recorded at 20, 30, 50, and 60 minutes. If an anchor fails in creep, re-testing should not be allowed.
3. The ground anchor should carry the maximum test load with less than 0.04 inches of movement between 1 and 10 minutes, or less than 0.08 inches between 6 and 60 minutes if the hold period is extended.
4. Total movement at the maximum test load should exceed 80% of the theoretical elastic elongation of the unbonded length.
5. When a ground anchor fails, the contractor should modify the design of the anchor, construction procedures, or both. These modifications may include, but are not limited to, installing replacement ground anchors, modifying the installation methods, increasing the bond length or changing the ground anchor type. Any modification that requires changes to the structure should have prior approval of the engineer.

Upon completion of the proof test, the load should be adjusted to the design lock-off load and transferred to the anchorage device. After transferring the load, and prior to removing the jack, the load should be checked by reapplying the load and measuring the lift off reading. This should be within 10% of the specified lock-off load. The process should be repeated until the desired lock-off load is obtained.

5.3.4 Monitoring and Instrumentation

Vertical and lateral movement of the ground surrounding the shored excavation is anticipated to some extent. Even with a well-designed shoring system there is a risk of greater than anticipated movement and possible damage to surrounding facilities. Therefore, a program of survey monitoring is recommended before and during the construction. This program should include measurements of the horizontal and vertical movement of the shoring, surrounding streets, buildings, and other facilities if necessary. Survey points should be established at the top of the shoring and at lateral distances equal to 1/3 H (where H is the depth of the excavation) and H perpendicular to the excavation. Survey monitoring points at the top of the shoring wall should be spaced no greater than 25 feet around the excavation. Several survey points should also be established on and in the vicinity buildings located adjacent to the site. The number of survey points and frequency of measurements should be determined based on the configuration of the final shoring design, and site and project constraints. As a minimum, during the excavation phase of the project, weekly measurements are recommended. The survey monitoring should be accurate to at least 0.01 foot for both vertical and horizontal distances.

In order to establish the condition of adjacent facilities prior to construction, a complete inspection and evaluation of pavements, structures, utilities, and other structures near the perimeter of the excavation should be performed. Any existing signs of damage, particularly those caused by settlement or horizontal movements, should be documented by photographs, notes, survey, drawings, or other means of verification.

5.4 Earthworks

Following the removal of existing structures/foundations and utilities, the site earthwork will include excavation, preparation of the building foundation and slab subgrades, backfilling foundation walls, and installing new utilities.

Based on our explorations, excavation for the proposed development would likely encounter uncontrolled fill overlying the native alluvium. Excavations should be sequenced to limit the amount of exposed soil and subgrade.

5.4.1 Subgrade and Foundation Preparation

Based on observations made during the site investigation, the soil at the proposed subgrade will likely consist of compact to dense alluvium soils. Foundations should be placed on the native alluvium; foundations should not be placed on uncontrolled fill.

5.4.1.1 Removal of Uncontrolled Fill or Other Unsuitable Soils

If encountered, uncontrolled fill or other unsuitable soils should be removed from beneath foundations. Uncontrolled fill and disturbed or loose native alluvium soils may be left in place beneath floor slabs and pavements if it can be compacted to a firm and unyielding condition as noted in Section 5.5.3.

5.4.1.2 Subgrade

Exposed subgrades for footings, floor slabs, pavements, and other structures should be compacted with a vibratory roller to a firm, unyielding state. Any localized zones of loose granular soils observed within a subgrade should be compacted to a density appropriate for planned development. Any organic, soft, or pumping soils observed within a subgrade should be removed and replaced with a suitable structural fill material. Unsuitable excavated materials should not be mixed with materials to be used as structural fill.

5.4.2 Use of Onsite Excavated Soil

The native alluvium soil is considered suitable for reuse as structural fill provided it can be placed and compacted near the optimum moisture content and in accordance with the compaction requirements presented in Section 5.4.3.3 of this report. If density tests indicate that compaction is not being achieved due to moisture content, the reused material should be scarified and moisture-conditioned to near optimum moisture content, re-compacted, and re-tested, or removed and replaced.

5.4.3 Structural Fill

The term "structural fill" refers to any materials placed under foundations, floor slabs, pavements, backfill for walls, and utility trench backfill. Golder's conclusions and recommendations concerning structural fill are presented in the following sections.

5.4.3.1 Materials

Where needed, structural fill should be free of organic and inorganic debris, be near the optimum moisture content, and capable of being compacted to the required specifications for application. Soils used for structural fill should generally not contain any organic matter or debris or any individual particles greater than 6 inches in diameter depending on use. Typical structural fill materials include clean sand and gravel; well-graded mixtures of sand and gravel (commonly called "gravel borrow" or "pit-run"); mixtures of silt, sand, and gravel; crushed rock; quarry spalls; and controlled-density fill (CDF). If the onsite soils do not meet the above criteria, or cannot be reworked to a suitable condition, we recommend using imported granular fill consisting of imported, clean, well-graded sand and gravel, such as "Gravel Borrow" per Washington State Department of Transportation (WSDOT): 9-03.14(1) (WSDOT 2014). Other fill materials may be used with approval of the engineer.

If imported material is needed for filling during wet weather, the project specifications should include provisions for using imported, clean, well-graded sand and gravel, such as "Gravel Borrow" per WSDOT: 9-03.14, except that the percent passing the US No. 200 sieve should be no greater than 5%.

5.4.3.2 Placement

Fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, and each lift should be thoroughly compacted with a mechanical compactor. Any structural fill placed beneath footings should extend laterally outside of the footing base at a 1H:1V (Horizontal to Vertical) slope projected down and away from the bottom footing edge. In areas of thick structural fill (greater than about 3 feet) this requirement may be relaxed with engineer's permission.

5.4.3.3 Compaction

Using the Modified Proctor test (ASTM D1557) as a standard, we recommend that structural fill used for onsite applications be compacted to minimum densities presented in Table 5-1.

Table 5-1: Compaction Criteria

Fill Application	% Minimum Compaction
Building pad	95
Footing subgrade or bearing pad	95
Slab-on-grade floor subgrade and subbase	95
Retaining wall footing subgrade	95
Concrete slab subgrades	95
Asphalt pavement base and subbase	95
Asphalt pavement subgrade	95
Retaining wall backfill	90 to 95
Footing and stemwall backfill	90
Landscaped Areas	85

5.4.3.4 Subgrade Verification and Compaction Testing

All structural fill should be placed over firm, unyielding subgrades prepared in accordance with the recommendations in this report. The condition of all subgrades should be verified by the geotechnical engineer before filling or construction begins. Fill soil compaction should be verified by means of in place density tests performed per ASTM D6938 during fill placement so that soil compaction may be evaluated as earthwork progresses.

Pavement and foundation subgrades should be maintained in a well compacted state and protected from degradation prior to paving or placing concrete. Protection measures may include restricted traffic, perimeter drain ditches, or placement of a protective gravel layer on the subgrade. Disturbed or wet areas should be removed and replaced by suitably compacted structural fill.

5.4.4 Wet Weather Construction

Although feasible, earthwork construction during wet weather or rainy season will significantly increase costs associated with off-site disposal of unsuitable excavated soils, amount of dewatering needed to reach foundation elevations, increased control of surface water, and increased subgrade disturbance and need for soil admixtures, geotextiles, or rock working mats.

For fill placement during wet-weather site work, we recommend using soils that have fines content of 5% or less (by weight). Laboratory tests of select samples from the site investigation indicate the on-site soils typically have around 5% fines.

5.5 Temporary Slopes

Safe temporary slopes are the responsibility of the contractor and should comply with all applicable Occupational Safety and Health Administration (OSHA) and Washington Industrial Safety and Health Act

(WISHA) standards. Temporary, stable cut slopes less than 8 feet in height can generally be constructed using the following recommendations:

- Uncontrolled Fill and Native Alluvium – 1.5H:1V

As previously discussed, groundwater will likely be encountered during construction. If temporary cuts encounter groundwater seepage, they should be sloped at 2H:1V or flatter (as recommended by the geotechnical engineer at the time of construction) to prevent significant caving or sloughing. Temporary cuts in the looser granular materials are expected to have some raveling at the cut face. Temporary cut slopes in granular soils may need to be laid back flatter than 1.5H:1V if a change in material type or debris is encountered.

In the event that groundwater seepage is encountered during excavation, the contractor must install temporary drainage measures to protect the cut face and prevent degradation of the excavation area until permanent drainage measures can be constructed.

5.6 Utilities

Maintaining safe utility excavations is the responsibility of the utility contractor. The soil and groundwater conditions in the utility excavations will vary across the site. Excavations in the looser granular soils may cave easily, while other excavations may be difficult, as occasional boulders and cobbles may be encountered. As appropriate, trench shoring should be employed by the utility contractor.

5.7 Geotechnical Construction Monitoring

We recommend that a qualified geotechnical-engineering firm is on-site during critical aspects of the project. This would include observation of excavation; footing, slab, pavement, and subgrade preparation; placement of wall and footing drains, and placement and compaction of structural fills. The geotechnical engineer of record will perform the special inspection.

6.0 CLOSING

This report has been prepared exclusively for the use of Washington Real Estate Holding, LLC and their consultants and contractors for the project site. This report may be reviewed by bidders and/or contractors as it relates to factual data only. The conclusions and recommendations presented in this report are based on the explorations and observations completed for this study, conversations regarding the existing site conditions, and our understanding of the planned project. The conclusions are not intended, nor should they be construed to represent, a warranty regarding the project. They are included to assist in the planning and design process.

Judgment has been applied in interpreting and presenting the results. The soil and groundwater conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times. Variations in subsurface conditions outside the exploration locations are common in glacial environments such as those encountered at the site and in areas disturbed by human activities. Actual conditions encountered during construction may be different from those observed in and inferred from the explorations.

It has been a pleasure to provide consulting services to Washington Real Estate Holding, LLC on this project. If you have any questions, please call us at (425) 883-0777.

GOLDER ASSOCIATES INC.

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D1557 Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 feet-lbf/feet3 (2,700 kN-m/m3)) (2012)
D1586 Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils (2011)
D2216 Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (2010)
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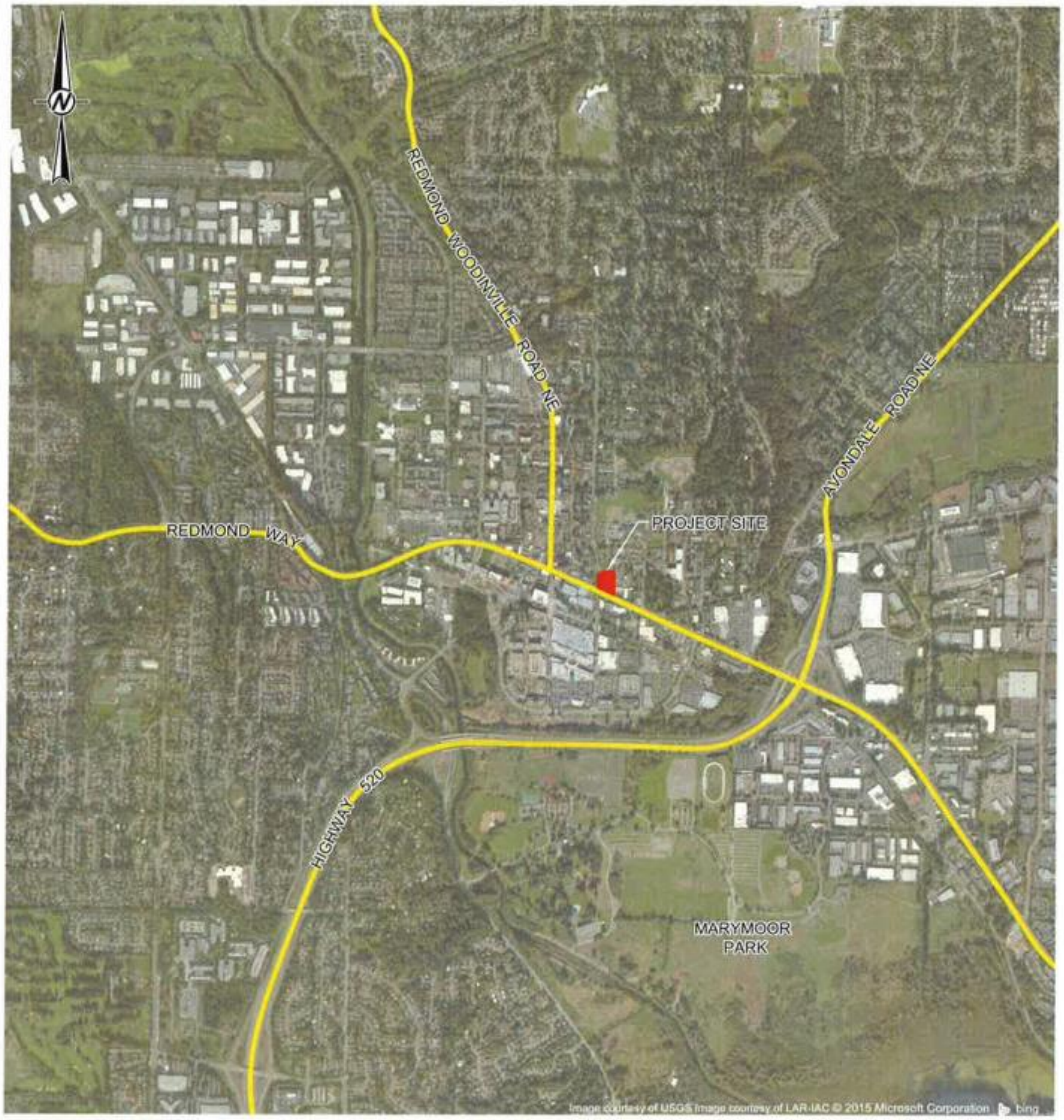
FIGURES



0 1000 2000
1" = 2000' FEET

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CLIENT
WASHINGTON REAL ESTATE HOLDINGS, LLC.

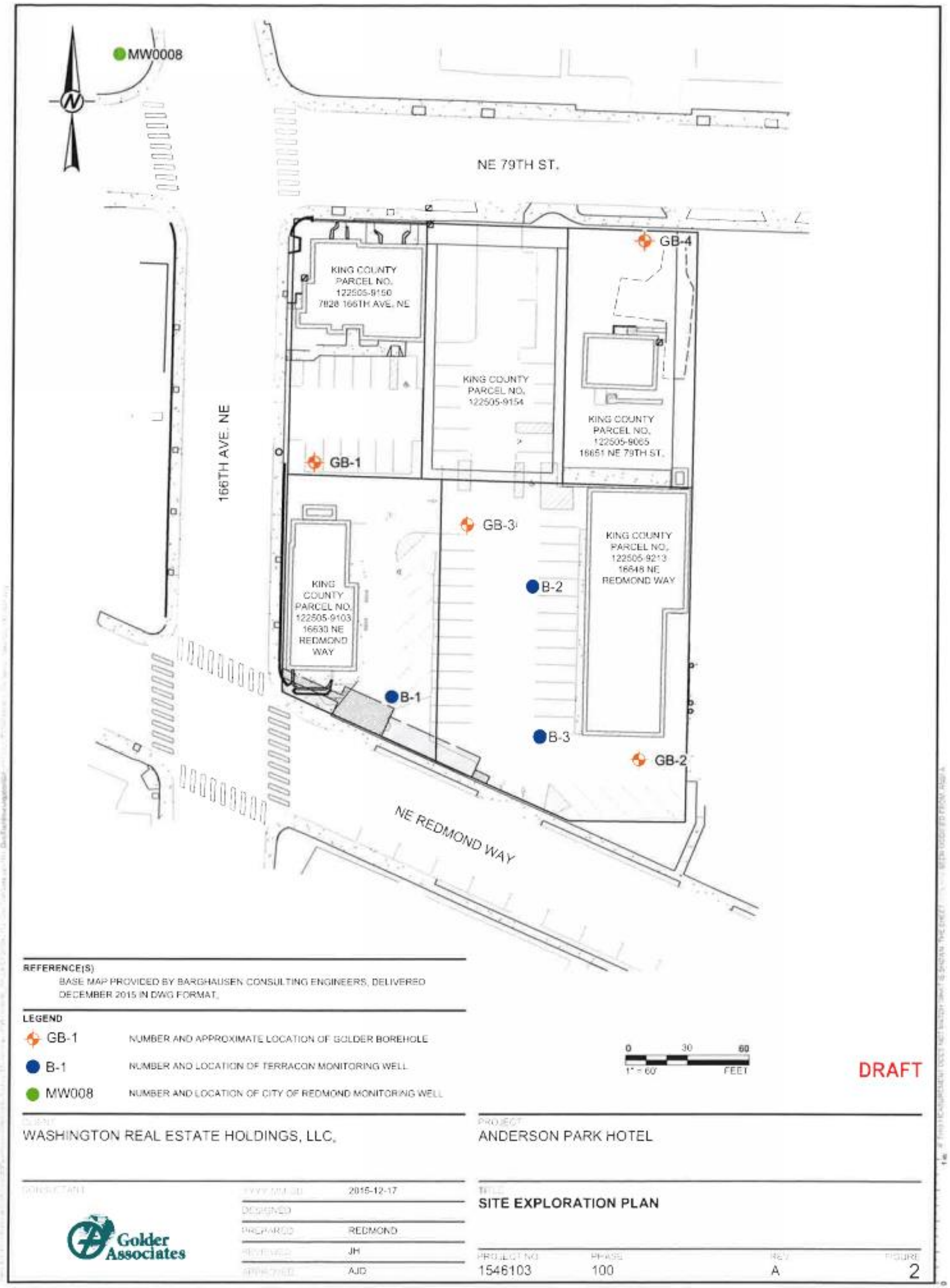
PROJECT
ANDERSON PARK HOTEL

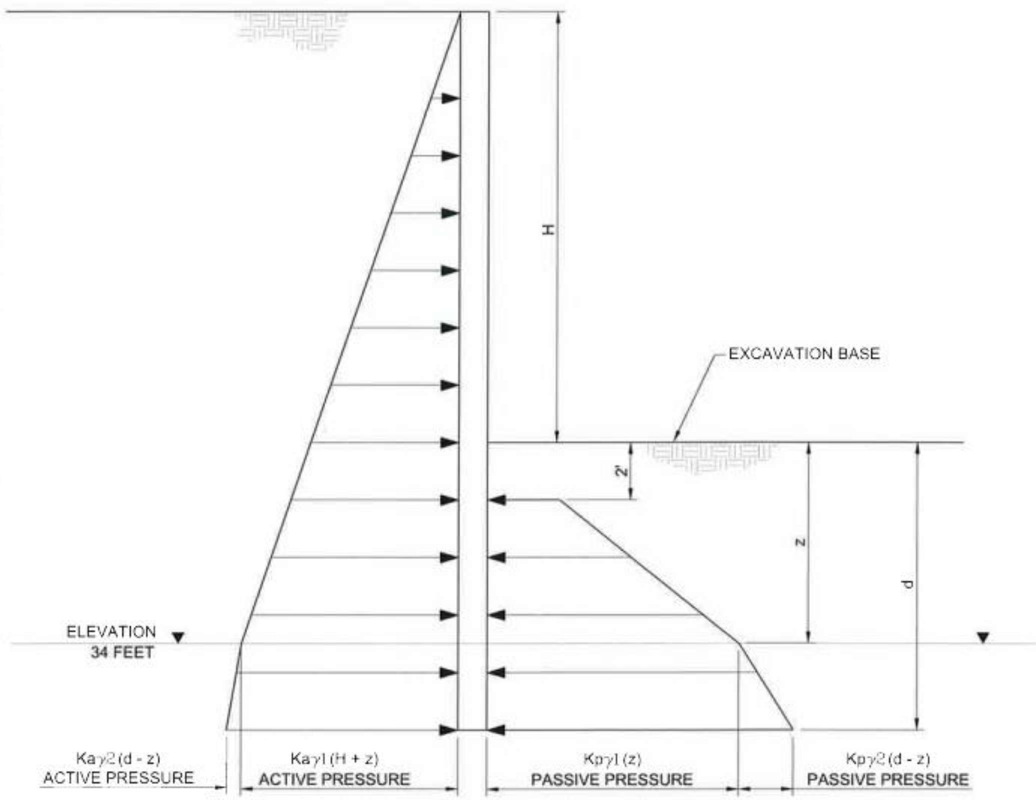
CONSULTANT
Golder Associates

YYYY-MM-DD	2015-12-17
DESIGNED	
PREPARED	REDMOND
REVIEWED	JH
APPROVED	AJD

TITLE
VICINITY MAP

PROJECT NO	PHASE	REV	FIGURE
1546103	100	A	1


MW0008
N
NE 79TH ST.
GB-4
KING COUNTY PARCEL NO. 122505-9150 7828 166TH AVE. NE
KING COUNTY PARCEL NO. 122505-9154
KING COUNTY PARCEL NO. 122505-9065 16651 NE 79TH ST.
166TH AVE. NE
GB-1
GB-3
B-2
KING COUNTY PARCEL NO. 122505-9213 16648 NE REDMOND WAY
KING COUNTY PARCEL NO. 122505-9103 16630 NE REDMOND WAY
B-1
B-3
GB-2
NE REDMOND WAY
REFERENCE(S)
BASE MAP PROVIDED BY BARGHAUSEN CONSULTING ENGINEERS, DELIVERED DECEMBER 2015 IN DWG FORMAT.
LEGEND
GB-1 NUMBER AND APPROXIMATE LOCATION OF GOLDER BOREHOLE
B-1 NUMBER AND LOCATION OF TERRACON MONITORING WELL
MW008 NUMBER AND LOCATION OF CITY OF REDMOND MONITORING WELL
0 30 60
1" = 60' FEET
DRAFT
WASHINGTON REAL ESTATE HOLDINGS, LLC.
PROJECT
ANDERSON PARK HOTEL
2015-12-17
SITE EXPLORATION PLAN
REDMOND
JH
AJD
Golder Associates
1546103
100
A
2



NOTE(S)

1. ALL DIMENSIONS IN FEET.
2. ALL PRESSURES IN POUNDS PER SQUARE FOOT (PSF).
3. PRESSURES ABOVE THE BASE OF THE EXCAVATION ACT OVER ENTIRE WALL FACE.
4. PASSIVE PRESSURE ACTS OVER TWO TIMES CONCRETED SOLDIER PILE DIAMETER, OR THE PILE SPACING WHICH EVER IS LESS.
5. SEE REPORT TEXT FOR RECOMMENDATIONS TO DETERMINE PILE EMBEDMENT AND VERTICAL CAPACITY.
6. SURCHARGE LOADS ARE NOT INCLUDED ON THE DIAGRAM.
7. ACTIVE AND PASSIVE PRESSURES ARE UNFACTORED.

LEGEND

H = WALL HEIGHT
D = EMBEDMENT DEPTH
z = DEPTH TO GROUNDWATER
$\gamma 1$ = TOTAL UNIT WEIGHT
$\gamma 2$ = SUBMERGED UNIT WEIGHT
Ka = ACTIVE EARTH PRESSURE COEFFICIENT
Kp = PASSIVE EARTH PRESSURE COEFFICIENT

DRAFT

WASHINGTON REAL ESTATE HOLDINGS LLC

ANDERSON PARK HOTEL

Golder Associates

YYYY-MM-DD	2015-12-22
DESIGNED	JLH
PREPARED	JLH
REVIEWED	SRV
APPROVED	AJD

EARTH PRESSURES DIAGRAM
CANTILEVER CONDITION OR ONE ROW OF TIEBACKS

PROJECT NO	PHASE	REV	FIGURE
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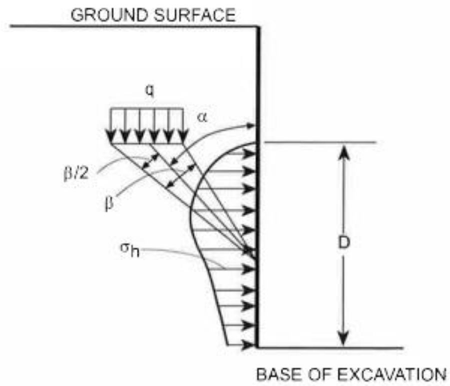
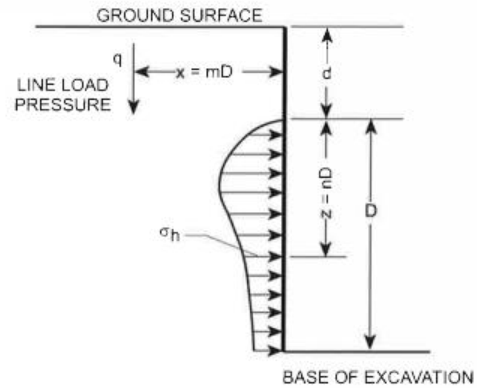


ISOLATED FOOTING

$\sigma_h = 0.64q\,(\beta - \sin\beta\cos 2\alpha)$



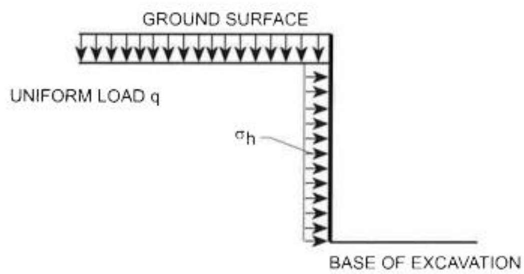
**CONTINUOUS FOOTING
PARALLEL TO EXCAVATION**

(For m > 0.4)

$$\sigma_h = \frac{1.28q}{D}\quad\frac{m^2 n}{(m^2 + n^2)^2}$$

(For m ≤ 0.4)

$$\sigma_h = \frac{q}{D}\quad\frac{0.2\,n}{(0.16 + n^2)^2}$$



UNIFORM LOAD DISTRIBUTION

σ_h = 0.2Q (ACTIVE CONDITIONS) OR
= 0.35Q (AT REST CONDITIONS)

q = VERTICAL PRESSURE IN PSF

DEFINITIONS & UNITS

D	EXCAVATION DEPTH BELOW FOOTING IN FEET
σ_h	LATERAL SOIL PRESSURE IN PSF
q	UNIT LOADING PRESSURE IN PSF
α β	RADIANS

DRAFT

CLIENT	WASHINGTON REAL ESTATE HOLDINGS LLC	
PROJECT	ANDERSON PARK HOTEL	
CONSULTANT	YYYY-MM-DD	2015-12-28
Golder Associates	PREPARED	REDMOND
	DESIGN	J.HANSON
	REVIEW	A. DENNISON
	APPROVED	J. JOHNSON
TITLE	LATERAL SURCHARGE PRESSURE ACTING ON BELOW GRADE WALLS	

PROJECT No.	PHASE	Rev.	FIGURE
1546103	100	0	4

APPENDIX D – INFILTRATION CALCULATIONS (WWHM)

WWHM2012
PROJECT REPORT

Project Name: Anderson Park Hotel
Site Name: Anderson Park Hotel
Site Address: 166th Ave NE
City : Redmond
Report Date: 10/11/2016
Gage : Seatac
Data Start : 1948/10/01
Data End : 2009/09/30
Precip Scale: 1.00
Version Date: 2016/02/25
Version : 4.2.12

Low Flow Threshold for POC 1 : 50 Percent of the 2 Year

High Flow Threshold for POC 1: 50 year

PREDEVELOPED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	**acre**
A B, Forest, Flat	**.64**
Pervious Total	**0.64**

Impervious Land Use	**acre**
Impervious Total	**0**
Basin Total	**0.64**

Element Flows To:

Surface	**Interflow**	**Groundwater**

MITIGATED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	**acre**

Pervious Total **0**

Impervious Land Use	**acre**
ROOF TOPS FLAT	**0.64**

Impervious Total **0.64**

Basin Total **0.64**

Element Flows To:

Surface	**Interflow**	**Groundwater**
Vault 1	Vault 1	

Name : Vault 1
Width : 90 ft.
Length : 8.5 ft.
Depth: 6 ft.
Infiltration On
Infiltration rate: 7
Infiltration safety factor: 1
Total Volume Infiltrated (ac-ft.): 100.501
Total Volume Through Riser (ac-ft.): 0
Total Volume Through Facility (ac-ft.): 100.501
Percent Infiltrated: 100
Total Precip Applied to Facility: 0
Total Evap From Facility: 0
Discharge Structure
Riser Height: 5 ft.
Riser Diameter: 12 in.

Element Flows To:

Outlet 1	**Outlet 2**

Vault Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.017	0.000	0.000	0.000
0.0667	0.017	0.001	0.000	0.124
0.1333	0.017	0.002	0.000	0.124
0.2000	0.017	0.003	0.000	0.124
0.2667	0.017	0.004	0.000	0.124
0.3333	0.017	0.005	0.000	0.124
0.4000	0.017	0.007	0.000	0.124
0.4667	0.017	0.008	0.000	0.124
0.5333	0.017	0.009	0.000	0.124
0.6000	0.017	0.010	0.000	0.124
0.6667	0.017	0.011	0.000	0.124
0.7333	0.017	0.012	0.000	0.124
0.8000	0.017	0.014	0.000	0.124

0.8667	0.017	0.015	0.000	0.124
0.9333	0.017	0.016	0.000	0.124
1.0000	0.017	0.017	0.000	0.124
1.0667	0.017	0.018	0.000	0.124
1.1333	0.017	0.019	0.000	0.124
1.2000	0.017	0.021	0.000	0.124
1.2667	0.017	0.022	0.000	0.124
1.3333	0.017	0.023	0.000	0.124
1.4000	0.017	0.024	0.000	0.124
1.4667	0.017	0.025	0.000	0.124
1.5333	0.017	0.026	0.000	0.124
1.6000	0.017	0.028	0.000	0.124
1.6667	0.017	0.029	0.000	0.124
1.7333	0.017	0.030	0.000	0.124
1.8000	0.017	0.031	0.000	0.124
1.8667	0.017	0.032	0.000	0.124
1.9333	0.017	0.034	0.000	0.124
2.0000	0.017	0.035	0.000	0.124
2.0667	0.017	0.036	0.000	0.124
2.1333	0.017	0.037	0.000	0.124
2.2000	0.017	0.038	0.000	0.124
2.2667	0.017	0.039	0.000	0.124
2.3333	0.017	0.041	0.000	0.124
2.4000	0.017	0.042	0.000	0.124
2.4667	0.017	0.043	0.000	0.124
2.5333	0.017	0.044	0.000	0.124
2.6000	0.017	0.045	0.000	0.124
2.6667	0.017	0.046	0.000	0.124
2.7333	0.017	0.048	0.000	0.124
2.8000	0.017	0.049	0.000	0.124
2.8667	0.017	0.050	0.000	0.124
2.9333	0.017	0.051	0.000	0.124
3.0000	0.017	0.052	0.000	0.124
3.0667	0.017	0.053	0.000	0.124
3.1333	0.017	0.055	0.000	0.124
3.2000	0.017	0.056	0.000	0.124
3.2667	0.017	0.057	0.000	0.124
3.3333	0.017	0.058	0.000	0.124
3.4000	0.017	0.059	0.000	0.124
3.4667	0.017	0.060	0.000	0.124
3.5333	0.017	0.062	0.000	0.124
3.6000	0.017	0.063	0.000	0.124
3.6667	0.017	0.064	0.000	0.124
3.7333	0.017	0.065	0.000	0.124
3.8000	0.017	0.066	0.000	0.124
3.8667	0.017	0.067	0.000	0.124
3.9333	0.017	0.069	0.000	0.124
4.0000	0.017	0.070	0.000	0.124
4.0667	0.017	0.071	0.000	0.124
4.1333	0.017	0.072	0.000	0.124
4.2000	0.017	0.073	0.000	0.124
4.2667	0.017	0.074	0.000	0.124
4.3333	0.017	0.076	0.000	0.124
4.4000	0.017	0.077	0.000	0.124
4.4667	0.017	0.078	0.000	0.124
4.5333	0.017	0.079	0.000	0.124
4.6000	0.017	0.080	0.000	0.124
4.6667	0.017	0.082	0.000	0.124
4.7333	0.017	0.083	0.000	0.124
4.8000	0.017	0.084	0.000	0.124

4.8667	0.017	0.085	0.000	0.124
4.9333	0.017	0.086	0.000	0.124
5.0000	0.017	0.087	0.000	0.124
5.0667	0.017	0.089	0.182	0.124
5.1333	0.017	0.090	0.509	0.124
5.2000	0.017	0.091	0.907	0.124
5.2667	0.017	0.092	1.318	0.124
5.3333	0.017	0.093	1.683	0.124
5.4000	0.017	0.094	1.960	0.124
5.4667	0.017	0.096	2.138	0.124
5.5333	0.017	0.097	2.300	0.124
5.6000	0.017	0.098	2.439	0.124
5.6667	0.017	0.099	2.571	0.124
5.7333	0.017	0.100	2.697	0.124
5.8000	0.017	0.101	2.817	0.124
5.8667	0.017	0.103	2.932	0.124
5.9333	0.017	0.104	3.042	0.124
6.0000	0.017	0.105	3.149	0.124
6.0667	0.017	0.106	3.252	0.124
6.1333	0.000	0.000	3.353	0.000

ANALYSIS RESULTS

Stream Protection Duration

Predeveloped Landuse Totals for POC #1
Total Pervious Area:0.64
Total Impervious Area:0

Mitigated Landuse Totals for POC #1
Total Pervious Area:0
Total Impervious Area:0.64

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.000543
5 year	0.000822
10 year	0.001049
25 year	0.00139
50 year	0.001687
100 year	0.002024

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Stream Protection Duration
Annual Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1949	0.000	0.000
1950	0.001	0.000
1951	0.001	0.000
1952	0.000	0.000
1953	0.000	0.000
1954	0.001	0.000
1955	0.001	0.000
1956	0.001	0.000
1957	0.001	0.000
1958	0.001	0.000
1959	0.001	0.000
1960	0.001	0.000
1961	0.001	0.000
1962	0.000	0.000
1963	0.000	0.000
1964	0.001	0.000
1965	0.000	0.000
1966	0.000	0.000
1967	0.001	0.000
1968	0.000	0.000
1969	0.001	0.000
1970	0.000	0.000
1971	0.001	0.000
1972	0.003	0.000
1973	0.001	0.000
1974	0.001	0.000
1975	0.001	0.000
1976	0.001	0.000
1977	0.000	0.000
1978	0.001	0.000
1979	0.000	0.000
1980	0.001	0.000
1981	0.000	0.000
1982	0.001	0.000
1983	0.000	0.000
1984	0.000	0.000
1985	0.001	0.000
1986	0.000	0.000
1987	0.000	0.000
1988	0.000	0.000
1989	0.001	0.000
1990	0.001	0.000
1991	0.001	0.000
1992	0.001	0.000
1993	0.000	0.000
1994	0.000	0.000
1995	0.001	0.000
1996	0.004	0.000
1997	0.001	0.000
1998	0.000	0.000
1999	0.001	0.000
2000	0.000	0.000
2001	0.001	0.000
2002	0.000	0.000
2003	0.000	0.000

2004	0.001	0.002
2005	0.000	0.000
2006	0.001	0.000
2007	0.005	0.000
2008	0.001	0.000
2009	0.001	0.000

Stream Protection Duration
Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.0053	0.0016
2	0.0036	0.0000
3	0.0026	0.0000
4	0.0010	0.0000
5	0.0010	0.0000
6	0.0008	0.0000
7	0.0008	0.0000
8	0.0005	0.0000
9	0.0005	0.0000
10	0.0005	0.0000
11	0.0005	0.0000
12	0.0005	0.0000
13	0.0005	0.0000
14	0.0005	0.0000
15	0.0005	0.0000
16	0.0005	0.0000
17	0.0005	0.0000
18	0.0005	0.0000
19	0.0005	0.0000
20	0.0005	0.0000
21	0.0005	0.0000
22	0.0005	0.0000
23	0.0005	0.0000
24	0.0005	0.0000
25	0.0005	0.0000
26	0.0005	0.0000
27	0.0005	0.0000
28	0.0005	0.0000
29	0.0005	0.0000
30	0.0005	0.0000
31	0.0005	0.0000
32	0.0005	0.0000
33	0.0005	0.0000
34	0.0005	0.0000
35	0.0005	0.0000
36	0.0005	0.0000
37	0.0005	0.0000
38	0.0005	0.0000
39	0.0005	0.0000
40	0.0005	0.0000
41	0.0005	0.0000
42	0.0005	0.0000
43	0.0005	0.0000
44	0.0005	0.0000
45	0.0005	0.0000
46	0.0005	0.0000
47	0.0005	0.0000
48	0.0005	0.0000

49	0.0005	0.0000
50	0.0005	0.0000
51	0.0005	0.0000
52	0.0005	0.0000
53	0.0005	0.0000
54	0.0005	0.0000
55	0.0005	0.0000
56	0.0005	0.0000
57	0.0005	0.0000
58	0.0005	0.0000
59	0.0004	0.0000
60	0.0004	0.0000
61	0.0004	0.0000

Stream Protection Duration
POC #1
The Facility PASSED

The Facility PASSED.

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0003	3048	2	0	Pass
0.0003	2731	2	0	Pass
0.0003	2391	2	0	Pass
0.0003	2145	2	0	Pass
0.0003	1920	2	0	Pass
0.0003	1696	2	0	Pass
0.0004	1480	2	0	Pass
0.0004	1334	2	0	Pass
0.0004	1179	2	0	Pass
0.0004	1009	2	0	Pass
0.0004	843	2	0	Pass
0.0004	680	2	0	Pass
0.0004	552	2	0	Pass
0.0005	446	2	0	Pass
0.0005	354	2	0	Pass
0.0005	258	2	0	Pass
0.0005	129	2	1	Pass
0.0005	36	2	5	Pass
0.0005	26	2	7	Pass
0.0005	26	2	7	Pass
0.0006	26	2	7	Pass
0.0006	26	2	7	Pass
0.0006	25	2	8	Pass
0.0006	25	2	8	Pass
0.0006	24	2	8	Pass
0.0006	23	2	8	Pass
0.0006	23	2	8	Pass
0.0007	23	2	8	Pass
0.0007	22	2	9	Pass
0.0007	22	2	9	Pass
0.0007	22	2	9	Pass
0.0007	21	2	9	Pass
0.0007	21	2	9	Pass
0.0007	21	2	9	Pass
0.0008	21	2	9	Pass
0.0008	19	2	10	Pass
0.0008	19	2	10	Pass

0.0008	18	2	11	Pass
0.0008	18	2	11	Pass
0.0008	17	2	11	Pass
0.0008	16	2	12	Pass
0.0009	16	2	12	Pass
0.0009	16	2	12	Pass
0.0009	16	2	12	Pass
0.0009	16	2	12	Pass
0.0009	15	2	13	Pass
0.0009	15	2	13	Pass
0.0009	15	2	13	Pass
0.0010	14	2	14	Pass
0.0010	13	2	15	Pass
0.0010	13	2	15	Pass
0.0010	13	2	15	Pass
0.0010	13	2	15	Pass
0.0010	13	2	15	Pass
0.0010	13	2	15	Pass
0.0011	12	2	16	Pass
0.0011	12	2	16	Pass
0.0011	12	2	16	Pass
0.0011	11	2	18	Pass
0.0011	11	2	18	Pass
0.0011	11	2	18	Pass
0.0011	11	2	18	Pass
0.0012	10	2	20	Pass
0.0012	10	2	20	Pass
0.0012	9	2	22	Pass
0.0012	9	2	22	Pass
0.0012	9	2	22	Pass
0.0012	9	2	22	Pass
0.0012	9	2	22	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0013	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0014	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0015	8	2	25	Pass
0.0016	8	2	25	Pass
0.0016	8	2	25	Pass
0.0016	8	2	25	Pass
0.0016	8	0	0	Pass
0.0016	8	0	0	Pass
0.0016	8	0	0	Pass
0.0016	8	0	0	Pass

0.0017	8	0	0	Pass
0.0017	8	0	0	Pass
0.0017	8	0	0	Pass

Water Quality BMP Flow and Volume for POC #1
On-line facility volume: 0 **acre-feet**
On-line facility target flow: 0 **cfs.**
Adjusted for 15 min: 0 **cfs.**
Off-line facility target flow: 0 **cfs.**
Adjusted for 15 min: 0 **cfs.**

LID Report

LID Technique	Used for Treatment?	Total Volumn Needs Treatment (ac-ft)	Volumn Through Facility (ac-ft)	Infiltration Volumn (ac-ft.)	Cumulative Volumn Infiltration Credit	Percent Volumn Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Vault 1 POC	N	91.53			N	99.92			
Total Volume Infiltrated		91.53	0.00	0.00		99.92	0.00	0%	No Treat. Credit

Compliance with LID Standard 8
Duration Analysis Result = Passed

Perlnd and Implnd Changes

No changes have been made.